TITLE:

Comprehensive Systems Analysis of Area-wide, Multi-modal Water Resources Utilization Systems(Dissertation_全文)

AUTHOR(S):

Okada, Norio

CITATION:

Okada, Norio. Comprehensive Systems Analysis of Area-wide, Multi-modal Water Resources Utilization Systems. 京都大学, 1977, 工学博士

ISSUE DATE:

1977-03-23

URL:

https://doi.org/10.14989/doctor.r3299

RIGHT:

京都大学学術情報リポジトリ
KURENAI 紅
Kyoto University Research Information Repository

工
369 函
1—0

Comprehensive Systems Analysis
of
Area-wide, Multi-modal Water Resources Utilization Systems

October, 1976

Norio Okada

Comprehensive Systems Analysis
of
Area-wide, Multi-modal Water Resources Utilization System

October, 1976

Norio Okada

Preface

This paper sheds light on some of the complex issues which beset water resources developments. The study is narrowed to several aspects of this many-faceted problem: area-wide, multi-modal water resources utilizations. The meaning of "water resources utilizations (developments)" is also limited to refer to the exploitation and use of water resource for water supply including its related regional activities. The notion of "area-wide" is used to refer to some forms of extensive utilizations such as inter-basin streamflow diversions (inter-basin water transfers) and cross-boundary municipal and industrial water supplies. The term "multi-modal" ("dual-modal") means some forms of conjunctive utilization of both fresh water developments of impounding surface water in dams and wastewater reclamations of recycling tertiarily treated waters for industrial uses.

By defining the problem as such, our preliminary focus will be placed on the evolution of some systematic breakdown of its multiple spectra by identifying the various kinds of system levels involved. In this manner the reader will be invited to the author's observation that some systems approach is a prerequisite to the treatment of the problem of this kind (Chapter 1).

On this basis this paper evolves six kinds of studies which will be approached by different types of systems analyses. These include: ①System Dynamics Approach to the Water Resources Management Related to the Regional Development (Chapter 2), ②Nonlinear Programming Approach for the Analysis of Intra-basin, Multi-modal Water Utilization System (Chapter 3), ③Decomposition-principle-based Analysis of Integrating Process of Inter-basin, Multi-modal Water Utilization System (Chapter 4), ④Nonlinear Goal Programming Approach for Co-ordinated Attainment of Multi-goal, Inter-basin, Multi-modal Water Utilization System (Chapter 5), ⑤Nonlinear Programming Approach for Analysis and Design of Water Distribution System (Chapter 6), and ⑥Systems Analysis of Operational Control of Water Supply and Use System in Drought-time (Chapter 7).

The paper closes with overall assessment of the major findings obtained in the six studies (Chapter 8).

Acknowledgment

For the original suggestion that such problems as covered here might be timely, the author is indebted to several people. Amongst them the author is much indebted to Professor Kazuhiro Yoshikawa for his valuable comments during the studies included in this paper and his critical review of the manuscript. Without his contribution the preparation of this paper would have been impossible. Also, the advice and encouragement of Assistant Professor Mamoru Haruna and Assistant Koshi Yamamoto have been of great assistance. Special thanks go to Professors Yoshiaki Iwasa, Takuma Takasao, Assistant Professors Hisashi Sumitomo and Shuichi Ikebuchi. The author has enjoyed the unfailing assistance of the members of the Laboratory of Systems Analysis and Planning In Civil Engineering, Department of Civil Engineering, Kyoto University. Especially the author is appreciative of those aids and cooperations given by Messrs. K. Yoshinaga, T. Wakatsuki, T. Mochizuki, T. Ohuchi, H. Sakurai, H. Nagano, Y. Iwasaki, H. Watanabe, and Y. Yagi. It is a pleasure to acknowledge the continued data services provided by the Ministry of Construction and Prefecture of Hyogo. The authors handwritten manuscript was expertly typed by Mrs. Miyako Takemoto.

Contents

List of Tables

List of Figures

Chapter 1 Introduction

1.1 Scope of the Paper

Water is the most basic of all our natural resources, a vital necessity for the growth and development of a city or a region. It is therefore a man's serious concern to find the ways of making best use of this resource. Man's different water-related activities of its efficient exploitations include (1) water supply, (2) power generation, (3) navigation, (4) recreation, etc.

Another facet of water takes the form of disadvantageous influences on man's life. The water-related activities to mitigate the factors detrimental to man concern (5) flood protection, (6) water-pollution alleviation, etc.

Although water reveals itself in such a multi-facet manner, this paper does not attempt to enter into a full coverage of all the features, but confines its scope of study to the problems of water supply with an incidental focus being placed on those of water-pollution alleviation (control), by taking account of the growing concern for a conjunctive treatment of the two problems when water supplies are discussed. Later, further references will be made to make the point.

1.2 Problems Treated

Even if the treatment of this study has roughly be specified as such, the problem remains still complex. Before we begin to consider some systematic break-down of the complex problem, let us devote some discussion to the identification of several types of water utilization problems which will lie within the scope of this study with prior reference to the status-quo of the related water problems in Japan.

1.2.1 Salient Features of Natural Water Resources of Japan

When the water resources of Japan are envisaged as a natural sources of water provisions, the following observations might be important.[1)2)]

(i) Available water resources are not necessarily affluent, judged from the physical and economical viewpoints, although the average annual rainfall seems to be relatively plentiful as compared to those of the other countries.

(ii) One simple criterion to judge the natural water availability is the per-capita rainfall. In terms of this criteria Japan is seen to rank relatively low among the countries in the world. (See Table 1.2.1.)

(iii) Daily or monthly stream discharges are subject to sharp fluctuations, chiefly owing to the river profiles as characterized by their sharp changes in elevations together with relatively narrow drainage areas. This is also the case with annual average stream discharges, which are fluctuating year by year on account of variation in annual average rainfall. The above

Name of Country	Rainfall (mm)	PerCapita Rainfall (m^3/yr/cap)
U.S.A.	833	39,200
U.K.	803	3,560
France	759	8,320
W.Germany	807	3,470
Italy	983	5,660
Canada	790	385,000
Norway	1,050	124,000
Spain	656	10,300
Sweden	700	40,000
Turkey	664	15,800
Holland	893	2,300
U.S.S.R.	390	37,100
China	837	11,100
India	1,224	7,830
Brasil	1,631	162,000
Japan	1,818	6,500

Table 1.2.1 Average Annual Rainfall of Different Countries

facts would provide another angle from which the low availability of nutural water resources of Japan can be viewed .

(iv) That is to say that they provide a salient picture of the typical pattern of water exploitations in Japan that the difficulty has been circumvented to some extent by throwing dams across the valleys in the headwaters of streams or by constructing other types of reservoirs on both the midstreams and downstreams.

(v) The geographical morphology of the land of Japan is not favorable for the costruction of dams in the sense that there are few number of those sites in Japan which would provide the required storage or elevations at a relatively small expenditure of labor and materials. For instance the minimum requied volume of concrete per a unit amount of storage experienced in Japan amounts to be ten to thirty times larger than that of many of the foreign countries.

(vi) Furthermore what makes the matter complex is that in Japan the inundation of the land after the completion of a dam often means the inundation of a village. This entails the compensation problem, which is becoming too entangled to find a compromise between the local residents and the agency responsible for the project. The needed expenditures for this kind of compensation are also increasing. In consequence this fact, coupled with the increased difficulty of finding suitable dam sites with an inexpensive cost, tends to place the construction of dams at the cost disadvantage year by year.

(vii) Available water resources are quite unevenly distributed. The grossest division would be to specify the highly urbanized regions as the area of low availability. They include the metropolitan districts centering on Tokyo, Yokohama and Chiba, and those highly industrialized districts whose centers are Osaka and Kobe. On the contrary we might roughly classify the sparsely-populated districts as the area of high availability, such as Hokkaido, Tohoku (northern parts of Japan), Sanin and Shikoku (western parts of Japan), etc. (See Table 1.2.2.)

1. 2. 2 Characteristics of Water Supply and Use in Japan

The water supply and use in Japan are characterized by the following factors.[1)2)]

Name of Part of Japan	Rainfall (mm)	PerCapita Rainfall (m³/yr/cap)
Hokkaidō	1,205	18,204
Tōhoku	1,820	12,701
Kantō	1,593	2,486
Hokuriku	2,890	13,108
Tōkai	2,390	5,924
Kinki	1,959	3,065
Sañin	2,110	16,052
Sanyō	1,770	6,728
Shikoku	2,130	10.223
Kyūshū	2,241	7,791

Table 1.2.2 Average Annual Rainfall of Different Parts of Japan

(i) The expanding industrial and commercial activities plus the growing populations in the urban areas are increasing water demands on conventional sources, chiefly the impounded water by reservoirs as will be hereafter called "fresh water". According to the future demands projected by the Ministry of Construction, the industrial and domestic water demands would be almost doubled in both the Tokyo Metropolitan and Kinki Districts by 1985.[1)] (See Table 1.2.1.)

(ii) In Japan agriculture accounts for very high percentages of total water withdrawals and consumptive uses as shown in Table 1.2.2. Stated otherwise, most of the low-flow discharge which is defined as that amount of stream discharge which can be utilized consistently all the year, has

Name of District	1960 (a)				1985 (b)				Increased Demand (b-a)			
	D*	I**	A***	sub-total	D	I	A	sub-total	D	I	A	sub-total
Hokkaidō	0.36	0.96	4.97	6.29	0.86	3.24	5.69	9.79	0.50	2.28	0.72	3.50
Tōhoku	0.91	1.61	12.95	15.47	1.75	4.65	14.38	20.78	0.84	3.04	1.43	5.31
Kantō	3.40	3.30	8.33	15.03	6.90	6.58	8.75	22.23	3.50	3.28	0.42	7.20
Hokuriku	0.20	1.02	2.84	4.06	0.46	2.06	2.89	5.41	0.26	1.04	0.05	1.35
Tōkai	1.06	3.87	4.64	9.57	2.71	6.42	5.28	14.41	1.65	2.55	0.64	4.84
Kinki	2.12	2.84	4.45	9.41	3.97	4.63	4.28	12.88	1.85	1.79	0.17	3.47
Sañin	0.10	0.18	1.32	1.60	0.16	0.43	1.38	1.97	0.06	0.25	0.06	0.37
Sanyō	0.45	1.43	3.56	5.44	1.10	2.65	3.84	7.59	0.65	1.22	0.28	2.15
Shikoku	0.26	0.90	2.31	3.47	0.62	2.28	2.64	5.54	0.36	1.38	0.33	2.07
Kyūshū	0.71	1.35	6.99	9.05	2.13	4.14	9.42	15.69	1.42	2.79	2.43	6.64
TOTAL	9.57	17.46	52.36	79.39	20.66	37.08	58.55	116.29	11.09	19.62	6.19	36.90

(Unit: Billion m^3/yr)

* D=Domestic Demand　　*** A=Agricultural Demand
** I=Industrial Demand　　(Source: Reference 1)

Table 1.2.3 **Predicted Water Demands in Different Parts of Japan**

already been exploited and traditionally utilized for agricultural uses. It is often the case that the water withdrawal of this kind has traditionally been regarded as "a right to water use". The transfer of agricultural water to domestic and industrial ones, which has long been advocated by the disinterested party as a promising remedy for water-deficit problems, will not be discussed in this paper, because (1) at the present stage it seems quite difficult to specify how much water is really used by the agricultural sector, including the amount of return flow of water finding its way back to some surface or ground water course. (2) Thereby it seems quite important and still difficult to distinguish the amount of water used from that consumed. (3) Another reason is that the transfer of such a right to water use to domestic and industrial uses would involve not only the institutional but political examinations —— too complicated and far-reaching to be covered by this study.

(iii) As has already been explained before, however, available sources for fresh water can hardly be found in the headwaters located in a relative proximity of industrialized areas, but only exploitable in the inmost recesses of that stream, on the downstream of which is located the water-demanding area.

(iv) But it seems also true that if we continue to shape our thinking to comform to traditional water use patterns —— utilizing water within a boundary of a given water basin (intra-basin water utilization) or resorting exclusively to the fresh water developments, we may continue to produce inadequate alternatives for water resources development. The point will be clear from the following observations.

1.2.3 Inter-basin Water Transfer

One type of prescription for alleviating these conditions is the inter-basin transfer of water from one river basin to another, or from one region of the country to another.[3)4)5)] A promising advocacy of this prescription is that it would provide fresh water more efficiently than it is, in the sense that ① the development costs could be reduced on the area-wide basis, and ② the redistribution of natural water resources which are unevenly distributed could be achieved from conveying water from a low-water demand region endowed with plentiful fresh water resources to a high water demand region with limited available sources.

The rebuttal provided by others points out that ① it only leads to the furtherance of the current water uses which have, are, and will tend to the increased industrialization and urbanization as well as the polluted streams in the urban areas, and that ② such a large-scale development of water resources could not be realized without destructing or degrading the natural environment. This kind of criticism is based on the understanding that population and industry should come where water is rather than sending water where the population now is.

1.2.4 Comprehensive Approach on Regional Basis

In connection with the above criticism the author takes the position that before entering into the explicit treatment of inter-basin water transfers, which are to constitute one of the key topics in this paper, the following questions must be asked and analyses should be made to put them into a systematic perspective. ① Is it not propitious to control the growth of population and/or industrial activities? ② If the answer is yes, then in what occasions and to what extent? ③ It must also be questioned —— what its impacts would be on the regional economy concerned? ④ If one sticks to the development of fresh water resources, would it really be impossible to manage to cover the water demand within the framework of that river basin in which the water-demanding regions are located?

The first and second questions could be imbedded into the third question, because this question cannot be answered without referring to the former questions. In light of these considerations, Chapter 2, the subsequent chapter, exhibits a systems analysis of the impacts of water resources management patterns on the regional activities as well as the backward impacts of the contracted activities on the water supply conditions. Thereby the reader will be invited to the observation that this kind of water resource problem must be examined within the framework of a regional analysis —— a broader comprehensive scope than that of the conventional studies which have usually treated the problem as a discrete entity —— isolated from the related interfaces.

1.2.5 Inter-basin Water Resources Development

The specific concern of the forth question stems from the basic understanding that ① in principle water problems should be managed to be solved within the limits of the concerned basin, because ② in a strictly physical sense of the word a single basin is considered a geographic unit of overland drainage contributing surface runoff to the flow of a particular stream or watercourse at a given point, and ③ in this context it has physically set limits to, and traditionally characterized the geographical basis for water-related activities. Furthermore ④ even if some form of inter-basin water transfer is to be undertaken in the last analysis, the maximum potential of intra-basin development should receive prior consideration. Otherwise it could not get an acceptance by the residents of a water-export area.

The above-cited considerations have led the author to the understanding that in prior to the detailed analysis of inter-basin transfers it is also necessary to give due consideration to the problem of intra-basin development. With this understanding, Chapter 3 will be devoted to the treatment of this kind of problem.

1.2.6 Wastewater Reclamation

Another kind of countermeasures are also receiving increasing publicity and public attentions —— some potential alternatives to inter-basin transfers or the regional growth regulations. One promising way is to develop other types of sources such as wastewater reclamation including on-site reuses by industries, desalting, groundwater development. Another alternative is the furtherance of increased efficiency in water use such as economizing of the demand side, reinforced works for the reduction in water-conveyance losses, and transfers from agriculture to municipal (domestic) and industrial uses. Excluding the increased efficiency in agricultural water use which will not be touched upon as explained before, the most promising source among the remedy alternatives is water reclamation which is characterized by the recycling reuse of renovated wastewaters, judged from the needed costs and available maximum scale of implementation. Two salient pitfalls can be pointed out, however. One concerns some hygienic troubles attendant with the quality of renovated water as an alternative source for drinking water. More often than not its quality is found to be inadequate for some specific industrial processes that require high quality of water provided. The other point is that even if it proves to be fit for drinking or other domestic uses as will become the case sooner or later, depending on the expected technological innovations in near future, the psychological refusal of water users to accept the method would deny a water supply utility its implementation. For this reason it will be postulated in this paper that water reclamation should be confined to industrial uses.

1.2.7 Urban Water Pollution

The disposal of the city's waste poses one of the most critical environmental problems of urban living. The dense urban population and substantial industrial and commercial economy, generate an enormous concentrated load of wastewater and solid wastes, which must be treated or transported out of the area. The collection and treatment of wastewater is an even more critical determinant of the quality of the urban environment, because wastewater treatment has traditionally relied on the dilution and assimilation of wastes by local surface water without prior treatment.

One effective way that is receiving a growing concern of practitioners and researchers is to undertake some water reclamation which is characterized by a high standard treatment process called tertiary treatment.[7] To this end also water reclamation is considered an effective tool.

1.2.8 Multi-modal Water Utilization

To conclude the above discussions the author will take the position that in finding the best ways of developing water resources to meet the future demands, "either/or" dichotomies are falling, and the focus should be on continuous spectra and not extremes. In other words an optimal mix of the two extreme alternatives —— fresh water development and water reclamation deserves due attention. This type of utilization will hereafter be called "multi-modal (dual-modal) water utilization". This kind of problem will be discussed in Chapters 3 to 5.

1.2.9 Area-wide Municipal and Industrial Water Supply

A set of water supply (utilization) facilities can be considered a system which is composed of a number of subsystems managed by different levels of agencies —— national, regional, or local level. These subsystems include dams and inter-basin diversion channels to be developed, managed mostly by some agency of national or local level, and municipal and industrial water supply facilities to be operated by a local-level agency as is often called a water supply utility. It is also assumed that both the prior treatment facilities which are usually known as wastewater plants and the tertiary treatment facilities constitute a water reclamation system which is also operated by a local-level agency. With particular reference to the latter systems managed by a local-level agency, the following problems will also be given close consideration.

(i) How can the overall water supply needs of a particular area or region be met in the most economical and practical manner? The answer to this question is dictated by comprehensive planning and its influence on a logical service area determination. The author will take the position that water supply is a service and a commodity which must be viewed on a broader base in order to meet immediate and future needs economically and effectively. For the best interests of all concerned, these cities and towns must be willing to accept the responsibility of providing water supply not only within their boundaries but also to the surrounding metropolitan areas. In this regard someone might contend that such a cross-boundary system would blur the scopes of responsibility taken by the party concerned. We shall assume that such a difficulty could be got around by instituting a third sector responsible for the management of the system as is often the case with the practical management systems. With this understanding, some explicit or implicit assumption will be made that both the municipal and industrial water supply system and the reclamation system to be incorporated into the total water utilization system are implemented on a cross-boundary basis.

(ii) With a specific focus on this local-level system two kinds of studies are conducted. One concerns the optimal implementation of a water distribution system which constitutes an extremity subsystem of the water utilization system. Chapter 6 will deal with this problem. The other will be discussed in the next paragraph.

1.2.10 Drought-time Water Supply Control

It goes without saying that a radical cure of the future water deficit is to formulate a long-range plan to develop different water sources within a context of regional planning. But another attention needs to be paid to the fact that some form of temporary measure should also be taken to cope with the present-day water crisis by the name of "water droughts". Furthermore since we have no external measures on hand in its exact sense of the word, such kind of adaptability at the level of operation seems to be of vital importance at any rate. From this point of view optimal operational control policies for alleviating unacceptable damages of droughts to the water users will receive explicit consideration in Chapter 7.

1.2.11 Water Quality Control

As has already been touched upon in 1.2.8, one of the most serious water resources problems facing most of the cities and towns and their peripheral areas today is the maintenance of satisfactory water quality levels in the rivers and lakes representing its fresh water and estuarine resources. The detailed analysis of water pollution mechanisms will not lie within the scope of this study,[7)] since our primary objective of its inclusion is to deal with it as one of the components of the water utilization system. That is, it is assumed in this paper that this mechanism is incorporated into the reclamation subsystem.

1.2.12 Coordination Problems

In dealing with the inter-basin, multi-modal water utilization system in Chapters 4 and 5, it gives rise to two types of coordination problems.[6)] Namely, Chapter 4 treats the coordination of two different planning functions associated with the selections of the most appropriate alternatives of both the fresh water development and reclamation systems. The interest of this analysis comes about from the observation that the former function goes to the jurisdiction of some agency of a national or regional level, whereas the latter to that of a local-level agency.

Another spectrum of the coordination problem involved in the inter-basin, multi-modal water utilization system is the reconcilation of the muliple goals involved, which will be analyzed in Chapter 5. The prescribed goals are maximized attainments of ① economic efficiency, ② acquisition of the needed amount of supply, and ③ stream water quality conservation, as have already been specified in an implicit manner in the foregoing discussions.

1.3 Systems Approach Applied

At the final stage of this chapter we shall sum up the preceding discussion with an intention ① to make a systematic breakdown of the problems treated and ② to reveal the necessity of systems approach which will constitute the basic methodology of this study.

1.3.1 Systematic Taxonomy of Problems Treated

1) System Levels

The complex problems of water utilization can be seen in perspective from several levels, which will be referred to as "system levels". The most simple way of giving an outlined image of this concept will be to present such polar distinctions as "macro" versus "micro", "coarse" versus "fine", and "general" versus "specific". These terms are used almost synonymous. To borrow the words of G.W. Reid[8], in systems formulation, there is a reciprocity between detail and variables, of finess and scope. The macro level is of course detail, but of a wide variety of variables. As details of finess are increased, variables must be reduced. Thus the micro system deals with water in detail as opposed to general commitment to other factors.

Another kind of dualisms typical to water utilization problems are such as "inter-basin" versus "intra-basin" systems, "cross-boundary" versus "intra-boundary" water supply systems, and "multi-modal" versus "single-modal" utilization systems.

Furthermore from the viewpoint of the scope of dominant functions to which the analysis is referred, those levels of "regional activities", "facility functions", "construction" and "operation" are considered. Thereby according to the difference in the commitment to the physical and practical conditions, the level of "facility functions" is broken down into two levels, that is, "facility planning level" and "design level". In a similar sense of the word, the level of "regional activities" might well be called "regional planning level". Alternately with reference to the scope of jurisdiction involved, "national", "regional", and "local" levels can be identified.

The notion of system levels might be broadened to include the dichotomies of "static" versus "dynamic", and "time-invariant" versus "time-variant" with reference to the mode of system formulation. The former dichotomy is sometimes identical with the latter, the basic difference being that the former is used to identify whether interactive functions are incorporated into the formulated system. The latter notion is used to clarify whether the factor of "time" is implicitly or explicitly treated in the model.

In this connection the reader is invited to the observation that irrespective of the manner time is treated, "time scale" should be identified in prior to the analysis. For instance if the problem concerns the operational control in drought-time, as will be dealt with in Chapter 6, the time scale of "one day", "one week" or at largest, "one month" would be appropriate. On the contrary when it concerns the discussion of courses of action for the water resources management in future, say, twenty to fifty years ahead, as will be treated in Chapter 2, then "one year" to "five years" seems to be fitted. We shall refer to this kind of level as "time-scale level".

As a matter of fact, however, the scales involved cannot be confined to "time", but various scales of inputs and outputs should be specified beforehand. To take an example of Chapter 3, the units of measurement scales such as water demands, stream discharges, BOD loads, construction and operation costs and so forth, need to be predetermined. In this sense of the word, the term "scale level" or "degree of finess" will be introduced.

At any rate the choice of a set of these system levels should be dependent on what should be treated and what needs to be obtained from the analysis. It is also subject to the data availability.

2) Taxonomy

The concept of different system levels as introduced above seems to give us an illustrative bird's-eye view of the diversified interfaces of the problem. On this basis systematic specifications of the problems treated in this paper will be attempted here. For further detail see Table 1.3.1.

Chapter 2 is devoted to the problem of long-range water utilization management whose specific interest is to discuss some possible courses of actions at the level of "regional activity." The selected time scale is "one year." Since it focusses on the interaction between the water-related and regional activities, a dynamic approach by the name of "systems dynamics", will be applied to the problem.

Chapter 3 deals with the "intra-basin", "multi-modal" development system which will be identified with a "time-invariant" one. The focus will be on the analysis at the level of "facility functions". The selected time scale is "one year".

In Chapters 4 and 5 the problems of the "inter-basin", "multi-modal" system which is defined as a "time-invariant" one, will be discussed with a view to provide some basic information on the policy-making at the level of "facility functions". The selected time scale is also "one year".

The central question to which Chapter 6 is devoted is the analysis of optimal implementation of the water distribution system defined as a "time-invariant" one. It exclusively concens those levels of "design", "cross-boundary supply", and "local".

Finally Chapter 7 analyzes the optimal measures to be applied to the drought-time control at an "operation" level, which is strictly related to the characteristics of some "local" water users. Two of the three studies included in this chapter deal with some "dynamic" mechanisms of drough-time water supply (and use) controls. The selected time scales are "one month" for the former and "one day" for the latter, according to the difference in the characteristics of the structured problems.

1.3.2 Mathematical and Simulation Models

The above systematic considerations well illustrate the systems approach to be used in this paper. The systems approach is strictly defined here as a set of systems analyses which are well arranged to present some well-structured information on the desired courses of action for the specified complex problems.

Basic to the systems analysis is a model, whether mathematical or simulation. The model is a symbolic representation of a real life situation. Its great advantage is the potential of that type of operational handling of its parameters which is called "sensitivity analysis" or "parametric programming". This kind of device is significant since the values of the parameters which are required to be identified in prior to the calculations on the model involve some degrees of "uncertainty". Some additional analyses are needed to gauge the impact of a possible change in the parameter values.

The above consideration has led the author to the observation that mathematical and simulation models are basic to the analysis of any one of the specified water problems. With this view much space of this paper will be spared for the descriptions of the model-building and its operational handling. Basic to a mathematical model is a solution search technique, without which it would make no sense to formulate it, because it could not be operated at all. In likewise a simulation technique is a prerequisite in the sense that the model cannot be simulated without it.

A model, whether mathematical or simulations will be constructed, much account taken of its correspondence to the required, "degree of finess". (See Table 1.3.1.) If it is concerned with some design level as in Chapter 6, refined physical and economic functions related to a specific locality will be incorporated into the model. Thus formulated functions exhibit a mold of nonlinearity. If the problem concerns a framework-making at a planning level, as is the case with Chapter 4, the model will be built in a linear form. In case the problem becomes more complex, some form of nonlinearity will be incorporated into the model.(Chapters 3 and 5) Suppose it deals with a time-variant problem, then a multi-stage mathematical model, i.e., dynamic programming model, will be formulated as in Chapter 7. If the problems are more complicate and deny us our attempt to find sophisticated mathematical relations involved, then we shall resort to a simulation model. Since in Chapter 2 our focus is placed on the "dynamic" tie-ins of water-concerned activities to regional ones, system dynamics will be applied to the problem. Similarly since the last of the three studies included in Chapter 7 deals with the interactions between the water supply and demand sectors, the same simulation technique will be used.

Chapter	System Level					
	Basin	Modality	Jurisdiction	Planning	Scale	Methodology & Formulation
II	Intra-basin	Multi-modal	Regional	Regional Activity	1 to 5 yrs	S.D. Time-variant
III	Intra-basin	Multi-modal	Regional & Local	Facility Planning	1 yr	N.P. Time-invariant
IV	Inter-basin	Multi-modal	Regional & Local	Facility Planning	1 yr	Decomposition Time-invariant
V	Inter-basin	Multi-modal	Regional & Local	Facility Planning	1 yr	N.P. Time-invariant
VI	Intra-basin	Single-modal	Local	Facility Design	1 yr & 1 month	N.P. Time-invariant
VII	Intra-basin	Single-modal	Local	Operation	1 day to 7 days 1 to 5 months	M.Q.M.,D.P.,S.D. Time-variant

S.D.=System Dynamics, N.P.=Nonlinear Programming, D.P.=Dynamic Programming, M.Q.M.=Multidimensional Quantification Method

Table 1.3.1 System Levels Involved

References

1) Ministry of Construction: Survey Report on Long-term Water Resource Management Perspectives, 1973 (In Japanese).
2) Water Resources Research Institute of Japan: Water Economics Annual Bulletin, 1975 (In Japanese).
3) Aki K., Tada H., et al.: Water Resources Handbook, Asakura Shoten Publ. Comp., 1972 (In Japanese).
4) Howe, C.W., Daster. K.W.: Inter-basin Transfers of Water, The John Hopkins Press, 1971.
5) Hartman, L.M., and Seastone, D.: Water Transfers, Economic Efficiency and Alternative Institutions, The John Hopkins Press, 1970.
6) Krutilla, J.V., and Eckstein, O.: Multiple Purpose River Development; Studies in Applied Economic Analysis, The John Hopkins Press, 1958.
7) Kneese, A.V., and Bower, B.T.: Managing Water Qualty; Economics, Technology, Institutions, The John Hopkins Press, 1971.
8) Reid, G.W.: The Systems Approach, Treatise on Urban Water Systems, Edited by M.L. Albertson et al, Colorado State University, 1971.

Chapter 2 System Dynamics Approach to the Water Resources Management Related to the Regional Development

2.1 Introduction

In Japan rapid increase in water demand has caused a serious scarcity of water in many parts of the country and the pressure on supply of usable fresh water is becoming more difficult. In these circumstances there has been a growing fear that water shortages might impede further growth of regional activities. In the "Survey Report on Long-term Water Resource Management Perspectives"[1] prepared by the Ministry of Construction of Japan, it is cautioned that water demands might grow largely in excess of suppliable water by the year of 1985. This caution seems to deserve close attention, but it should also be noted that this perspective is mainly based on the premise that the conventional growth pattern of economic and social activities as well as water use patterns will still remain unchanged in a couple of decades. Stated otherwise, if the economic and water use patterns are modified, water demands might be reduced. This means that the extent of water shortage is dependent on the assumed regional growth patterns, and the term "water shortage" does not necessarily mean the "absolute stringency of water".

2.2 Scope of the Study

The primary objective of this chapter is to build up a model that views water resource management as the major source of concern and that will serve as an effective tool to see the role of water within the context of regional development.[2] It is desired to see the effect of regional activities upon the water demand and supply systems, and at the same time to gauge the impact of the alternative systems of water resource management upon the regional activities. Our approach to the problem is to construct a simulation model with system dynamics[4)5)6)7)] for the reasons that follow.

(i) System dynamics is well qualified for taking explicit account of the dynamic interaction between regional and water-concerned activities, the analysis of which is the key objective of the study.

(ii) The special purpose compiler, which is called DYNAMO is available, which will reduce significantly the cost of programming and running the model. In addition it offers such advantages as easily understood model statements, easily specified outputs in both tabular and plot formats.

2.3 Regional Setting of the Model

The study area consists of two adjoining regions, i.e., the eastern part of the Harima Region (called "Tohban Region"), and the northern part of the Settsu Region (called "Hokusetsu Region"), both located along the southern coastal area of Hyogo Prefecture in Japan, and both forming parts of the Hanshin Industrial District. (See Figure 2.3.1.)

2.3.1 Hokusetsu Region

This region which boasts the Kobe International Port, one of the leading ports in Japan, is characterized by the highly integrated industrial and commercial activities which have been constantly maintained for a half of the

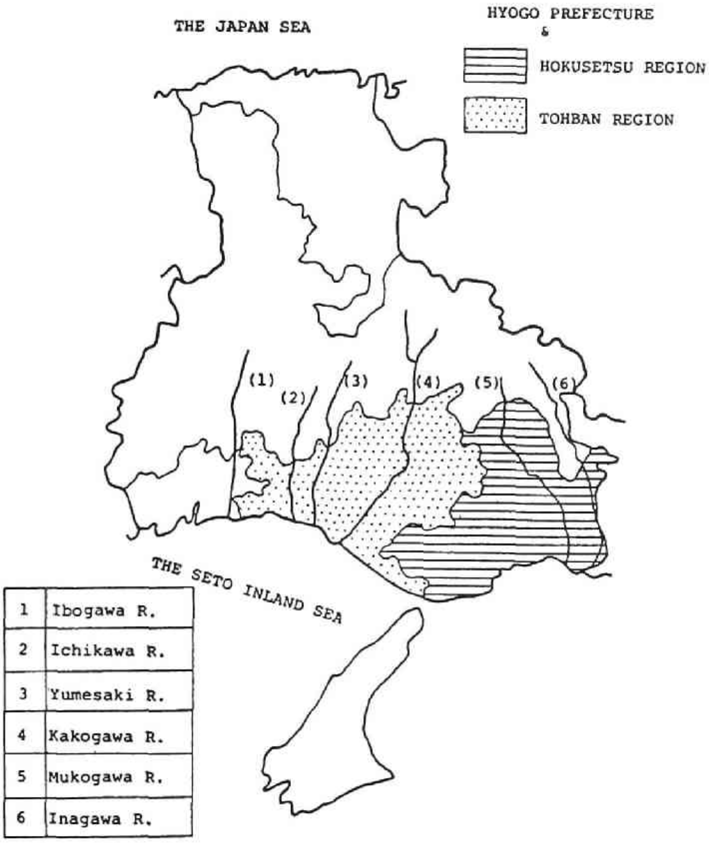


Fig. 2.3.1 Study Area

centuary. Such concentrations of economic activities have led the region to intensified deficiency of housing and transportation, magnified pollutants, increased stringency of water resource, etc. Some of the basic economic indices depicting the feature of the region are listed in Table 2.3.1.

2.3.2 Tohban Region

This region, located next to the Hokusetsu Region, has been growing rapidly and a series of large-scale reclamation works which have actively been promoted by the local government, and the Chuhgoku-jukan Highway, whose construction has recently been completed ---- these have lifted the region up to one of the important and active industrial districts both in the prefecture and the nation. These

REGION	MUNICIPALITY	AREA	POPULATION	IMMIGRANTS	EMIGRANTS	WORKERS (1972)		
						PRIMARY	SECONDARY	TERTIARY
HOKUSETSU	KOBE	539 (km²)	1,285,235	72,205	64,210	174,504	213,125	439,840
	AMAGASAKI	49	554,170	48,980	44,945	98,773	112,288	97,608
	NISHINOMIYA	96	374,715	43,025	35,290	25,000	33,610	62,798
	ITAMI	25	153,320	20,230	11,825	30,853	33,876	29,193
	TAKARAZUKA	102	127,065	17,795	11,295	7,752	10,159	18,479
	KAWANISHI	54	86,990	11,725	6,260	6,215	8,042	10,856
	SANDA	212	33,210	1,760	1,475	3,487	4,716	8,228
	TOTAL	1,077	2,614,745	215,720	175,300	346,584	415,816	667,002
TOHBAN	HIMEJI	268	408,745	21,640	17,825	75,294	94,142	115,232
	KAKOGAWA	98	126,470	16,570	5,755	21,841	26,601	23,445
	TAKASAGO	34	68,875	6,185	3,815	20,360	23,469	14,100
	TOTAL	400	604,090	44,395	27,395	117,495	144,212	152,777

Table 2.3.1 Basic Regional Figures of the Study Area

INDUSTRIAL WATER	WATER-CONSUMPTIVE INDUSTRIES		WATER-NON-CONSUMPTIVE INDUSTRIES	
	FRESH WATER	953,606 (m^3)	FRESH WATER	599,930 (m^3)
	REUSED WATER	577,746 (m^3)	REUSED WATER	240,448 (m^3)
	TOTAL	1,531,452 (m^3)	TOTAL	840,378 (m^3)
	REUSE RATIO	37.8 (%)	REUSE RATIO	37.4 (%)
DOMESTIC WATER	MUNICIPALITY	MAXMUM SUPPLY (m^3/day)	MAINLY WITHDRAWN FROM (DIRECT COLLECTION ONLY)	
	KOBE	534,891	River Hazuka, Ikuta & Ishii	
	AMAGASAKI	200,000	R. Yodo	
	NISHINOMIYA	124,422	R. Mukogawa	
	ITAMI	49,300	R. Inagawa	
	SANDA	5,500	R. Mukogawa	
	TAKARAZUKA	21,000	R. Mukogawa	
	KAWANISHI	8,545	R. Inagawa	
	TOTAL	943,658		

(Hokusetsu Region)

INDUSTRIAL WATER	WATER-CONSUMPTIVE INDUSTRIES		WATER-NON-CONSUMPTIVE INDUSTRIES	
	FRESH WATER	3,177,903 (m^3)	FRESH WATER	198,915 (m^3)
	REUSED WATER	2,492,492 (m^3)	REUSED WATER	64,686 (m^3)
	TOTAL	5,670,395 (m^3)	TOTAL	263,601 (m^3)
	REUSE RATIO	44.0 (%)	REUSE RATIO	24.3 (%)
DOMESTIC WATER	MUNICIPALITY	MAXIMUM SUPPLY (m^3/day)	MAINLY WITHDRAWN FROM (DIRECT COLLECTION ONLY)	
	HIMEJI	83,600	River Ichikawa (0.17m^3/s) R.Ibo (0.05m^3/s)	
	KAKOGAWA	20,000	—	
	TAKASAGO	30,580	R. Kakogawa	
	TOTAL	134,180		

(Tohban Region)

Table 2.3.2 Basic Water-Concerned Figures of the Study Area

features will be easily understood by the basic figures concerned with the regional activities in the region as listed in Table 2.3.1.

2.3.3 Status-quo of Water-concerned Activities

Fig.2.3.1 portrays the rives which run through the regions concerned or from which suppliable water is withdrawn. The Hokusetsu Region owes its major sources of water to Rivers Yodo and Muko. In the Tohban Region major sources are Rivers Kakogawa, Yumesaki, Ichikawa and Ibogawa. The numerical data concerning the sources of collected water for the industrial and domestic uses, are listed in Table 2.3.2.

So far as water supply and use activities are concerned, the absolute amounts of needed water in the Hokusetsu Region are relatively higher than others, and available water resources have approached the critical limit of development in a technological sense, and the probability of water shortage occurrences is growing higher and higher with the passage of time. Notably enough, these serious conditions involved in the water-concerned activities, have promoted water-savings which have practically been embodied in the forms of increased applications of on-site reuse systems of factories and reduced growth rate of the absolute amounts of consumed water. As to the water-concerned activities in the Tohban Region, this region is marked by a rapid growth rate of water consumption, tremendously high ratio of consumed water per product shipment and relative opulence of available water.

2.4 Model Building

2.4.1 Outlined Model Structure

The approach that was taken for building our model depends largely on following several over-all considerations.

(i) The study treats the impact of the water supply and use system upon the region, which is the primary objective but not the end of the analysis because thus influenced regional activities would also have an impact on the water supply and use system.

(ii) This means that, at a minimum, we need to incorporate into the model the structure of the performances of activities concerned with the water supply and use system. (This kind of activities will be called "water-concerned activities".)

(iii) In another view the incorporation of feedback loops corresponding to the interactions between regional and water-concerned activities is one of the most important prerequisites to the study.

(iv) As far as regional activities are concerned, we aimed at selecting exclusively those factors which seem to be dominantly related to and governing water demand generation and use patterns. In this view we selected two major factors dominating over the water-concerned activities: the demographic factor and industrial factor.

In light of these considerations two subregional models for the above-specified are constructed, each of which is modelled separately but in a similar fashion. These subregional models are composed of the three major sectors, i.e., water, demographic and industrial sectors.

2.4.2 Incorporated Feedback Loops

In view of above discussions some basic feedback links are incorporated between different sectors. In the simple illustration shown in Figure 2.4.1,

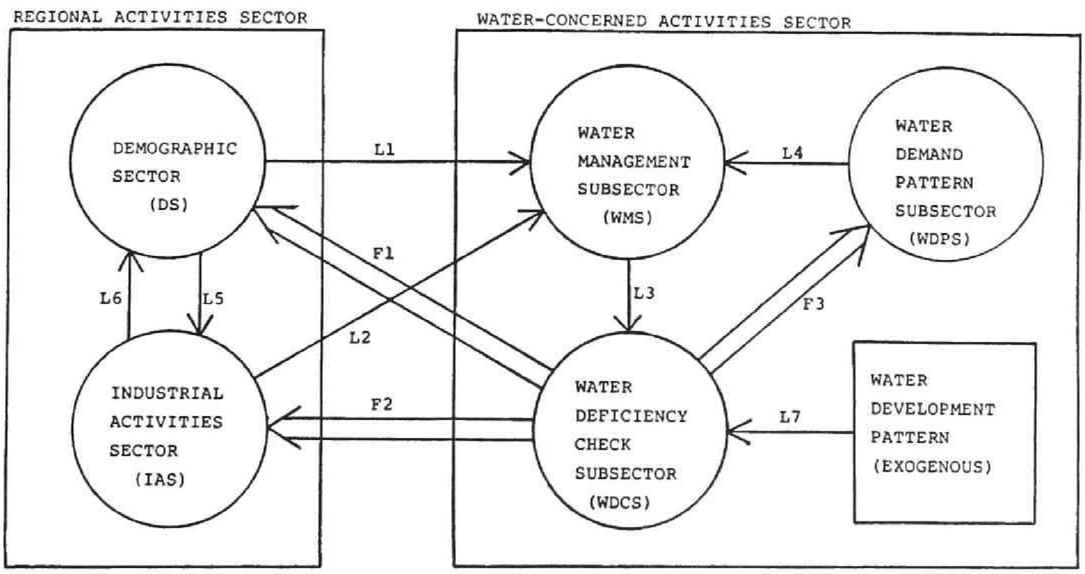


Fig. 2.4.1 Basic Model Structure

the arcs drawn from one circle to another shows how the variables concerned with each sector interact with the other sectors, where each circle represents a sector and bolded arcs the feedback links between sectors or sub-sectors. (See Figure 2.4.1.)

1) Feedback Link I (denoted by F1)

This link which is stemming from the "Water-Deficiency-Check Sub-sector" (denoted by WDCS) to the "Demographic Sector" stands for the alternative courses of action (policy) which automatically operates depending on the extent to which water shortages are critical. The "extent of water shortages" will be explained later. The "alternative courses of action (policy)" means a series of measures to limit the increase in population of the region in question.

As the incorporated feedback link constitutes a "feedback loop" combined with the flows as denoted by L1 and L3, it is also important to observe that whenever it acts, it will eventually affect other variables within or without the sector, directly or indirectly in the sense that the effects will come out immediately or with some time-lags, or in another sense that the impacts of the feedbacks operative depending upon the extent of water shortages will later return to the water-concerned activities themselves.

2) Feedback Link II (denoted by F2)

The second feedback link from the WDCS onto the IAS (the "Industrial Activities Sector") represents the alternative courses of action toward the IAS when water shortages come out. Conceptually it means the measures such that the growth rate of industrial activities are retarded so as to reduce the absolute amounts of needed water by industrial activities. The criterion to decide the degree of retardation totally depends upon the extent of water shortages in a similar fashion as F1, F2 and L2, L3 constitute a feedback loop.

3) Feedback Link III (denoted by F3)

The third one is found between two different subsectors of water-concerned activities: the WDCS and the WDPS (the Water Demand Pattern Sub-sector). This link is set to function in case water shortages occur. In other words it means the measures through which the water demand pattern is modified such as accelerated implementation of on-site reuse system or regional water reclamation systems. In likewise as the above, there is also a feedback loop comprising F3 and L4, L3, all of which are found in the "Water-concerned Activities Sector."

These three basic feedback loops are regarded as the "intersectoral feedback loops". It must also be noted that there are another type of feedback loops incorporated into each of the sectors and sub-sectors. These feedback loops which we refer to as the "sectoral feedback loops" will be explained later in the descriptions that follow.

2.4.3 Explanation of Time Notation and Variable Types

The basis for the time notation is the procedure by which the computer calculates the results, which is to move through TIME in discrete steps and calculate all the variables at each step. Figure 2.4.2 shows the procedure

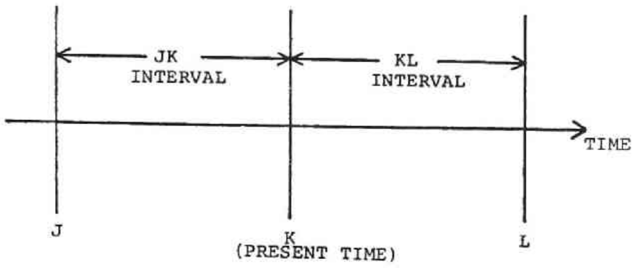


Fig. 2.4.2 Time Notation in DYNAMO

graphically. The TIME for which the calculations are currently being made is called TIME K. The previous time for which calculations were made is called J, and the next instant for which calculations will be made is L. The intervals between these times are called JK and KL. The length of these intervals is called DT.

There are three principal types of variables in DYNAMO: levels, rates, and auxilaries.

1) Level

A level, which is calculated at TIME K, is a quantity that depends upon its value at TIME J and on other quantities at that TIME or in the JK interval. Inventory is an example in that the inventory today is equal to the inventory at an earlier time plus what has been received minus what has been shipped during the interim.

2) Rate

The decisions in business and economic models are rates. Rates are the flows of tangible things from one level to the next. They are computed at TIME K for the interval KL from levels and auxiliaries at TIME K and occasionally from rates in the JK interval.

3) Auxiliary

Auxiliaries are variables that are introduced to simplify the algebraic complexity of rate equations. They generally have some physical meaning and consequently simplify the understanding of the model. They are computed at TIME K from TIME J and the JK interval. By their nature they can be eliminated by substitution into the rate equations.

2.4.4 Demographic Sector Submodel

1) Levels

The Demographic Sub-sector describes the growth of the population in a given region. In this view we set population as a level which was divided into seven classifications. (See Table 2.4.1.)

2) Rates

As shown in Table 2.4.1, we set: (1) the number of deaths (at the K-L interval; denoted by DYT), (2) the number of births (BBT), (3) the number of a given age classification (people) who grow out of the age-class into the next class (B⊔T), (4) the number of net movers (IMGL⊔T). This is defined as the absolute number of immigrants minus that of emigrants. In the above ⊔ represents one

VARIABLE TYPE	NOTATION	WHICH STANDS FOR:
LEVEL	AMT	TOTAL AREA OF INDUSTRIAL LANDS, WHERE ACTUAL PRODUCTIONS ARE BEING PERFORMED
	BAMT*1	INCREASED AREA OF INDUSTRIAL LANDS, DEVELOPED 1 YEAR BEFORE
	BAMT*2	2
	BAMT*3	3
	BLFT*1	NUMBER OF WORKERS LIVING WITHIN THE REGION, 1 YEAR BEFORE
	BLFT*2	2
	BLFT*3	3
	BET*1	NUMBER OF EMPLOYEES WORKING WITHIN THE REGION, 1 YEAR BEFORE
	BET*2	2
	BET*3	3
	AMIT2	REGULATORY POLICY CONCERNING INDUSTRIAL LAND DEVELOPMENT
RATE	NONE	—

Table 2.4.2 Levels and Rates for the Industrial Activities Sector

VARIABLE TYPE	NOTATION	WHICH STANDS FOR:
LEVEL	NWDT	WATER DEMAND
	AWST	SUPPLIABLE WATER
	SPUAT	INDUSTRIAL WATER DEMAND UNIT
	BWRT	PROPORTION OF REUSED WATERS TO THE TOTAL INDUSTRIAL WATER DEMAND
	PUPT	DOMESTIC WATER DEMAND UNIT
RATE	AWIT	INCREASING RATE OF THE TOTAL WATER DEMAND
	AWSIT	INCREASING RATE OF SUPPLIABLE WATER DEVELOPED
	PUADT	INCREASING RATE OF THE INDUSTRIAL WATER DEMAND UNIT

Table 2.4.3 Levels and Rates for the Water-Concerned Activities Sector

VARIABLE TYPE	NOTATION	WHICH STANDS FOR:
LEVEL	POPBT	POPULATION, AGE 0
	POPCT	, AGE 1 through 14
	POPHT	15 19
	POPYT	20 24
	POPAT	25 34
	POPMT	35 54
	POPOT	OVER 55
RATE	DYT	NUMBER OF DEATHS
	BBT	NUMBER OF BIRTHS
	BCT	NUMBER OF THOSE SHIFTING TO NEXT AGE-CLASS, ORIGINALLY BELONGING TO: AGES 1 THROUGH 14
	BHT	AGES 15 19
	BYT	20 24
	BAT	25 34
	BMT	35 54
	BOT	OVER 55
	IMGBT	NUMBER OF NET MOVERS, AGES 0
	IMGCT	AGES 1 Through 14
	IMGHT	15 19
	IMGYT	20 24
	IMGAT	25 34
	IMGMT	35 54
	IMGOT	OVER 55

Table 2.4.1 Levels and Rates for the Demographic Sector

of those letters, i.e., C,H,Y,A,M,O; corresponding to those age-classes: 1 through 14, 15 through 19, 20 through 24, 25 through 34, 35 through 53, over 54, respectively.

3) Sectoral Feedback Loops
(See Figure 2.4.1.)

4) Basic Equations Formulated

The number of people, ages 20 through 24, is a level formed by the rates of those growing into the age-class (BYT), and the number of net movers (IMGYT) less the number of those growing out of the age-class (BAT) and number of those dying (DYT):

POPYT.K=POPYT.J+(DT)(BYT.JK+IMGYT.JK-DYT.JK-BAT.JK) ·························(2.L·1)

Here BYT, for instance is a rate formed by POPYT, denoting the number of people, ages 15 through 19 (a level), and DRY, proportion of those dying to the age-classification people:

BYT.KL = 1/5(POPYT.K)(1-DRY) ·······································(2.R·1)

Other detail formulations of the equations concerned are omitted here and hereafter.

2.4.5 Industrial Activities Sector Submodel

The first problem in connection with modelling the industrial activities is selecting some appropriate ingredients which may well serve the dynamical system of the industrial activities in the concerned regions. The basic hypothesis implemented here, which we have arrived after close examinations of collected data concerned is that the dynamical change of the region's industrial activities depends primarily on the growth of industrial land areas and employment rate.

1) Levels

The level variables we set are as follows: ① AMT denoting the total area of industrial lands where actual productions are being performed, ② BAMT*1, BAMT*2, BAMT*3 denoting the box-car-train variables of the increased area of industrial lands developed one, two, and three years before, respectively; ③ BLFT*1, BLFT*2, BLFT*3 denoting box-car-train variables of the numbers of workers living within the region at the times of one, two and three years before, respectively; ④ BET*1, BET*2, BET*3 denoting the box-car-train variables of the number of employees working within the region at the times of one, two and three years before, respectively; ⑤ AMIT2 denoting the policy concerning the degree of limitation on the development of industrial lands. (See Table 2.4.2.)

Among these five kinds of variables, ②, ③ and ④ are dummy level variables in the sense that they are set only to store the box-car-train memories. AMIT2 is an extraneous level variable the value of which is predetermined as a policy variable. Notably, equations concerned with this sector are expressed only in terms of level and auxiliary variables, exclusive of rate variables.

2) Basic Equations Formulated

Equations concerning the change in the industrial land area are:

AMT.K = AMT.J + (DT)(BAMT*3J) .... (2.L·2)

This equation means that there assumed to be three year time lag between the completion and actual provision times of industrial lands, which proved to be reasonable through examinations on real world data.

AMIT.K = (AMIT1.K)(AMT.K) .... (2.A·8)

AMIT1.K = MT3 + (MT4)(ASRT.K) + (MT5)(LFIRT.K) .... (2.A·9)

Equation 2.A·9 represents the relation between the increasing rate of industrial land area (AMIT1), the proportion of the industrial zone area developed to the existing industrial land area (ASRT), and the increasing rate of the number of workers living within the region. MT3, MT4, MT5 are constant parameters whose values are predetermined by use of multi-regression analysis.

LFIRT is calculated with the following equations:

LFIRT.K = ELFIT.K/LFT.K .... (2.A·10)

ELFIT.K = 1/3(LFT.K - BLFT*3.K) .... (2.A·11)

where ELFIT denotes the number of the increased workers living within the region averaged over the preceding three years (auxilary variable), LFT the number of workers living within the region (auxilary variable), and BLFT the number of the same at the time of three years before.

Equations concerning alternative courses of policy associated with the degree of limitation on the development of industrial lands are:

AMIT1.K = MT3 + (MT4)(ASRT.K) + (MT5)(LFIRT.K) (2.A.9)

AMIT3.K = MAX(0,AMIT.K) .... (2.A·13)

AMDT1.K = MIN(0,AMIT1.K) .... (2.A·14)

AMIT5.K = (AMIT3.K)(AMIT2.K) .... (2.A·15)

AMIT4.K = AMIT5.K + AMDT1.K .... (2.A·16)

where AMIT1 denotes the increasing rate of the industrial land area corresponding to the case where the policy does not operate, AMIT4 the rate corresponding to the case where the policy operates, AMIT3, AMDT1, and AMIT5 are dummy auxiliary variables, incorporated to make the policy inoperative in case the industrial land area decreases. AMIT2 was set as a function of DWST. (See Figure 2.4.3.)

While the above set of equations represent the regulatory mechanism of the industrial land area in accordance with the degree of water shortages, we set another kind of regulatory mechanism which operates depending on the "extent of open space available for industrial lands." This extent is defined as the surplus of the space capacity of industrial lands (the industrial land limit) over that needed for further industrial land development. Its value was given a priori through close examinations on existing land use patterns and its future perspectives. The formulation of the mechanism is omitted here. Besides the above formulations, we incorporated into this submodel such set of equations as those concerning the change in the number of workers

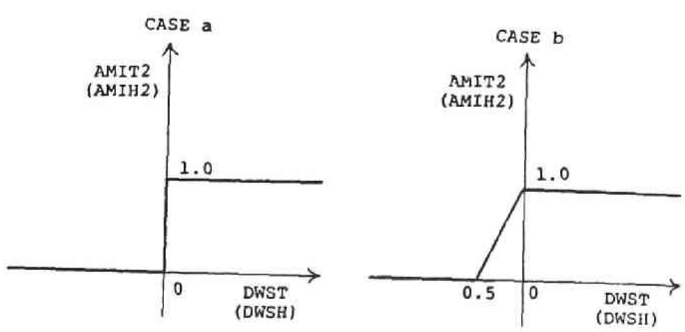


Fig. 2.4.3 Established Step Function with respect to DWST

living within the region, the change in the number of employees working within the region, and the change in the amount of production shipment.

2.4.6 Water-concerned Activities Sector Submodel

In modelling the water-concerned activities it is imperative to examine the following items: ① What should the "water demand" be strictly defined as? And what about the "suppliable water"? ② How to simulate the mechanism of water demand generation. ③ How to simulate the mechanism of suppliable water development. ④ How to incorporate into the model the mechanism of the alternative courses of policy, which are taken only when water shortages are critical. In connection with these problems, ① and ② are discussed here. The rest of the problems will be referred to later.

(1) Definition of the Water Demand and Suppliable Water

The water demand is defined as the amount of water substantially needed for both industrial and domestic activities, exclusive of agricultural activities. Here the "amount of water substantially needed (used, or consumed)" means the amount of needed water subtracted by the amount of reused waters. The water demand as thus defined was assumed to be supplied with any fresh water develope It was standarized by that of 1965, which we set as the starting point in the simulation runs. Therefore the water demand was set equal to those amounts of water needed for industrial and domestic activities which accumulate from that of 1965.

The suppliable water is defined as the amount of fresh water that is developed on and after 1965, and that owes its major sources of supply to the reservoirs developed on the rivers concerned.

In view of above considerations we incorporate into the submodel the following levels and rates.

2) Levels

① NWDT denoting the water demand, ② AWST the suppliable water, ③ SPUAT the industrial water demand unit, which is defined as the amount of water needed for a unit amount of industrial production shipment, ④ BWRT the proportion of reused waters to the total industrial water demand, ⑤ PUPT the domestic water demand unit, which is defined as the amount of water needed by one person.

3) Rates

① AWIT denoting the increasing rate of the total water demand, ② AWSIT the increasing rate of suppliable water developed, ③ PUADT the decreasing rate of the industrial water demand unit.

4) Basic Equations Formulated

We set five kinds of sets of equations, any of which are called a sub-sector: ① equations concerning the change in the water demand (the Water Demand Pattern Sub-sector), ② the change in the suppliable water (the Water Development Pattern Sub-sector), ③ the change in the water demand units (the Water Use Pattern Sub-sector), ④ the change in the reuse ratio of industrial water (the Water Use Pattern Sub-sector), ⑤ the change in the degree of water shortages (the Water

Shortage-Check Sub-sector). The brief descriptions of these equations will follow.

(i) The industrial water demand at time K is estimated with the following equations:

IWDT.K = (CT.K)(IWDT1.K) .... (2.A·41)

IWDT1.K = (AMST.K)(SPUAT.K) .... (2.A·42)

where IWDT denotes the industrial water demand, IWDT1 that of those industries with more than 30 employees, SPUAT the industrial water demand unit, CT the proportion of IWDT1 to IWDT, and AMST the value of shipments.

In likewise the domestic water demand at time K is estimated as:

WDT.K = (POPT.K)(PUPT.K) .... (2.A·43)

where WDT denotes the domestic water demand, POPT the total population, PUPT the domestic water demand unit.

Accordingly the total water demand is calculated as:

NWDT.K = IWDT.K + WDT.K - ULWT.K .... (2.A·44)

where ULWT denotes the water demand at the time of 1965.

(ii) Different alternative development patterns are extraneously set on the basis of data presented by the Ministry of Construction[1], which has closely examined possible development sites of dams and barrages, and estimated the potential amount of water whose development was assumed to be possible from technical and financial points of view.

The cumulative amount of suppliable water developed on and after 1965 is given by the following level equation:

AWST.K = AWST.J + (DT)(AWSIT.JK) .... (2.L·41)

(iii) The industrial water demand unit is estimated as:

PUAT.K = PUAT.J - (DT)(PUADT.JK) .... (2.L·42)

PUADT.KL = (PIRAT)(IWSRT.K)(PUAT.K - LLPT) .... (2.R·41)

where PUAT denotes the industrial water demand unit, PUADT its decreasing rate, LLPT its lower limit, IWSRT a modification factor, the value of which is dependent on alternative courses of policy to be taken according to the degree of water shortages, and PIRAT a predetermined constant.

The domestic water demand unit is calculated as:

PUPT.K = PUPT.J + (DT)(PUPIT.JK) .... (2.L·43)

PUPIT.KL = (PIROT)(WSRT.K)(ULPT - PUPT.K) .... (2.R·42)

where PUPT denotes the domestic water demand unit, PUPIT its increasing rate (given a priori), WSRT a modification factor whose value is dependent on the alternative courses of policy to be taken according to the degree of water shortages, ULPT the projected upper limit of the domestic water demand unit, and PIROT a constant value given a priori through multi-regression analysis applied to the real-world data.

(iv) The (on-site) reuse ratio of industrial water is defined as the proportion of the reused waters to the total water needed in cooling and temperature-controlling processes, where different types of reuse systems are practically implemented and further technological advances can be expected to scale up the systems whereas in other kinds of processes such as boiler-operation, scouring or those processes where high-quality of water is used as one of the raw-materials, reuse systems cannot be implemented in several decades to come.

In light of these considerations, the change in the reuse ratio of industrial water is given as:

$$WRT.K = MAX(BWRT.K, WRT3.K) \quad \cdots\cdots (2.A\cdot 51)$$

$$WRT3.K = WRT1.K + (WRT2.K)(1 - WRT1.K) \quad \cdots\cdots (2.A\cdot 52)$$

$$BWRT.K = BWRT.J + (DT)(WRT.J - BWRT.J) \quad \cdots\cdots (2.A\cdot 53)$$

where WRT denotes the reuse ratio of industrial water, WRT3 its actual ratio taken at the present time (K), BWRT its actual ratio taken in the preceding year (J), WRT1 the projected reuse ratio of industrial water, and WRT2 a policy variable whose value is given according to that of DWST.

Then the substantial amount of industrial water corresponding to the industrial water demand as defined before, is calculated as:

$$SPUAT.K = PUAT.K - TPUAT.K \quad \cdots\cdots (2.A\cdot 54)$$

$$TPUAT.K = (WRT.K)(CPUAT.K) \quad \cdots\cdots (2.A.55)$$

$$CPUAT.K = (CWRT.K)(PUAT.K) \quad \cdots\cdots (2.A.56)$$

where PUAT denotes the total industrial water demand unit, SPUAT its substantial demand, CPUAT the industrial water demand unit associated with cooling and temperature-controlling operations, TPUAT the industrial water demand unit covered by reuse systems, CWRT the proportion of the industrial water for cooling and temperature-controlling processes to the total water demand, the values of which are given a priori by extrapolating the real-world trends.

(v) Here we defined the degree of water shortages (DWST) as follows:

$$DWST.K = (AWST.K - ARWT.K)/(ARWT.K + ULWT) \quad \cdots\cdots (2.A.57)$$

where AWST denotes the suppliable water developed on and after 1965, ARWT the water demand in the year immediately preceding (J) minus that of 1965, ULWT the water demand of 1965.

We assumed that different alternative courses of policy are formulated as functions of DWST. The policy is associated with the following: 1 immigration control, 2 industrial location control, 3 water use pattern regulation. The detail descriptions in connection with this kind of policy have already been given in the preceding discussions.

2.5 Simulation of the Model

2.5.1 Preliminary Discussion

In keeping with our iterative research strategy, the simulations were made at different stages in the model's development in order to increase understanding of its dynamic behavior and determine which were its more sensitive parts. The results of these simulation experiments were then used for directing model improvement effort.

Since our approach has stressed taking explicit account of the dynamic interaction between water-concerned and regional activities, it is of our primary concern to shed light on the question of whether or not the interdependence between two different sectoral activities, plays an important role in the regional performance. With this kind of emphasis, the impact of changes in assumptions, data and policies was of major interest and the absolute values of levels generated by the model were of secondary interest. In this respect we made the best use of sensitivity experiments, which consist of making changes in the model, usually in the value of a particular parameter and compare the result simulated with the change to the result simulated without the change. This procedure is helpful, in identifying those parameters or aspects of the model that could make significant differences in the outputs.

In light of these considerations the following simulation-run cases were prepared to make different kinds of experiments.

(i) According to the difference in the values of those parameters, i.e., population upper limit (denoted by ULTO for the Tohban Region or ULHO for the Hokusetsu Region) and industrial land area upper limit (denoted by UATO or UAHO), three different cases were established. They are designated by Case I, Case II and Case III, respectively, amongst which the first is assumed to take the most moderate (or the most optimistic) upper limit values, whereas the last is designed to take the most severe (or the most pessimistic) values. (See Table 2.5.1.)

(ii) According to the difference in the water supply development pattern, 15 cases were considered. They are roughly classified into two categories. Case A where water supply development is assumed to be made at a slow but steady pace, Case B where it is assumed to be made rapidly but intermittently. Case B can be further classified into two cases: Case B-1 where all the developments are assumed to be made in the primary stage, Case B-2 where they are assumed to be made toward the end of the period.

(iii) According to the difference in the policy patterns associated with area, two different cases were set: Case a for a severer restriction policy and Case B for a moderate one.

(iv) According to the difference in water demand unit projections, we set Case α and Case β. Figure 2.5.1 illustrates the two kinds of projections of water demand units: the former (Case α) is generally adopted in most of the simulation-runs as standard cases; the latter applied in comparison with the standard cases.

(v) Simulations were conducted separately for two differect subregions: Case T

MAJOR CATEGORY	CASE I (STANDARD)				CASE II			CASE III			COMMENTS: KEY POINT	COMMENTS: FOR CLASSIFICATION
											UPPER LIMIT	I: severest II: medium III: moderest
SUB-CATEGORY	A		B		A	B		A	B		WATER RESOURCE DEVELOPMENT	A: slow and steady B: rapid and intermittent
			1	2		1	2		1	2		1: former-stage-integrated 2: latter-stage-integrated
	a		a·b	b	a	a	a	a	a	a	RESTRICTION POLICY	a: severer b: moderate
	α	β	α	α	u	α	α	α	α	α	WATER DEMAND UNIT PROJECTION	α: optimistic β: pessimistic
NUMBER OF CASES	3	1	3	1	1	1	1	1	1	1	NONE	
SUBTOTAL	9				3			3				
TOTAL	15										NONE	

Table 2.5.1 Simulation-Run Cases

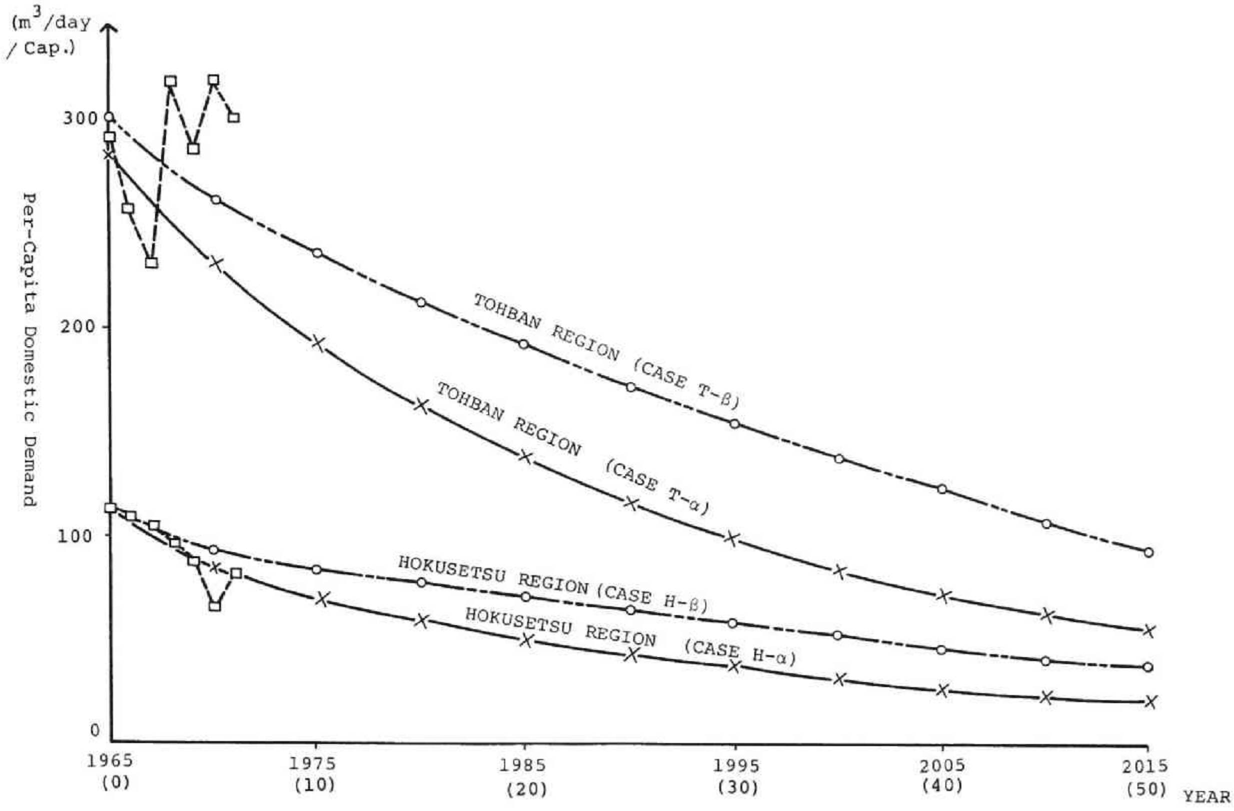


Fig. 2.5.1 Two Types of Projected Water Demand Units.

and Case H corresponding to the Tohban and Hokusetsu Regions, respectively.

The above five major classifications of the simulation-run cases were mutually combined to form the further ramified cases. For instance one of them is denoted by Case H-III-B-1-a-α.

So far as inputs are concerned, they are classified into two types: initial values and constant parameters. For instance DT is 1 year, the simulation time length 50 years, and the initial time is set at 1965. Some of the input values are listed in Table 2.5.2 and Figure 2.4.3 and 2.5.1 to 2.5.3.

2.5.2 Typical Model Runs

Before discussing the model experiemnts of diversified cases in detail, we will examine some typical model runs so as to have an overall view of the dynamic behavior of the model. Figures 2.5.4 and 2.5.5 are plots of several selected variables resulting from such simulations. From these figures it may be understood the following:

(i) Although water shortages occur on a small scale for a limited short period, suppliable water tends to be in surplus over that needed in a macroscopic view.

(ii) In other words the problem of water shortages may not fall into one of the critical events which could alter drastically the future growth pattern of regional activities in regions concerned.

(iii) The above may generally apply to either of the two subregions, whereas in the Hokusetsu Region water shortages occurrences and their degrees are apt to be higher than in the Tohban Region. This means that the Tohban Region is

	NOTATION	WHICH STANDS FOR:	WHICH TAKES THE VALUE OF:
SPEC.	DT	CALCULATION UNIT TIME INTERVAL	50(yrs.)
SPEC.	LENGTH	SIMULATION TIME LENGTH	1(yr.)
C*	ULTO	POPULATION UPPER LIMIT	3.38(mil.)
C	UATO	INDUSTRIAL LAND AREA UPPER LIMIT	6,530(ha)
N**	AMT	INDUSTRIAL LAND AREA	2,347(ha)
N	PUAT	INDUSTRIAL WATER DEMAND UNIT	281.4 (m^3/milyen)
N	PUPT	DOMESTIC WATER DEMAND UNIT	o.181 (m^3/milyen)
C	MT21	COEFFICIENT OF REGRESSIN EQUATION	0
C	MT22	THE SAME ABOVE	0.247
C	MT23	THE SAME ABOVE	1.18
N	POPBT	POPULATION, AGE 0	17.25

Table 2.5.2 Major Input Values

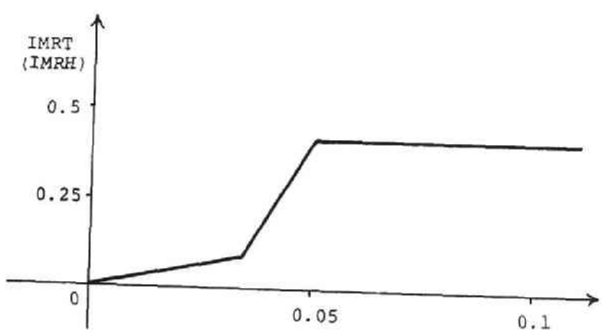


Fig. 2.5.2 Established Policies (1)

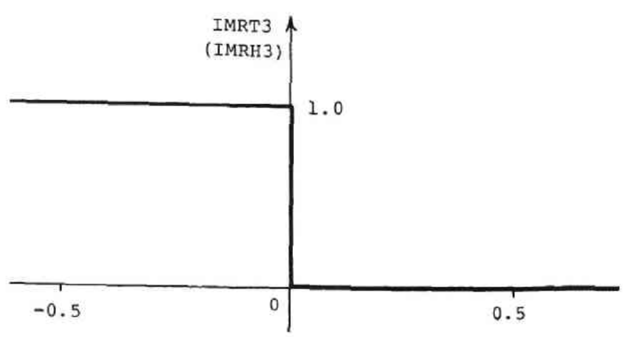


Fig. 2.5.3 Established Policies (2)

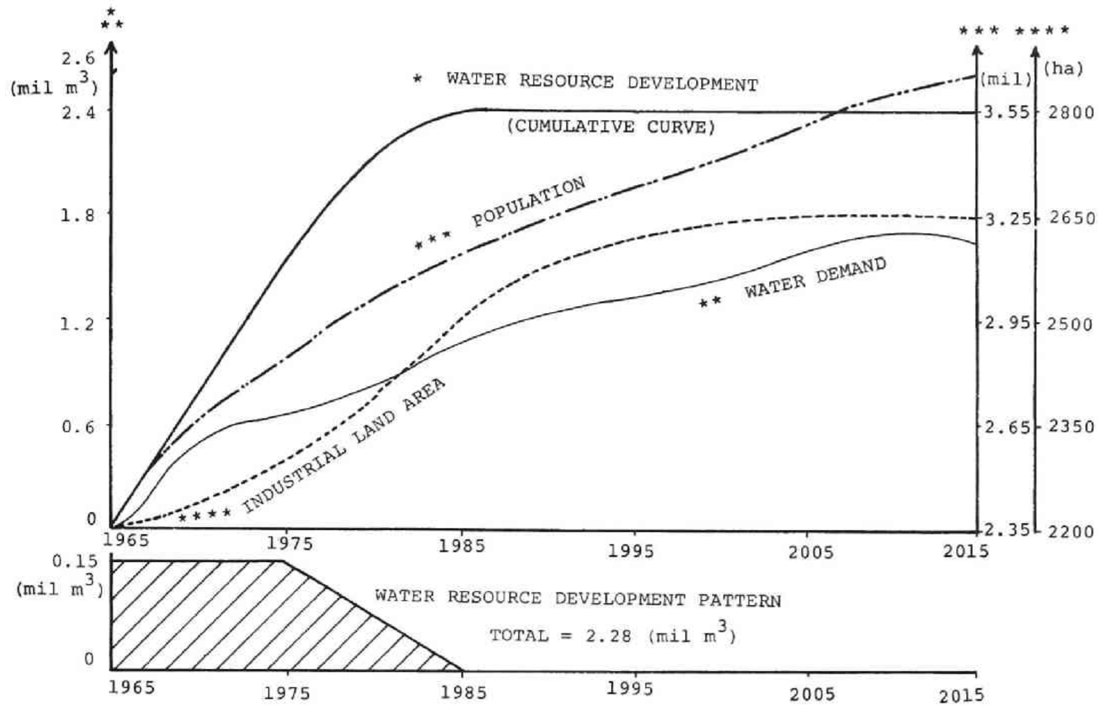


Fig. 2.5.4 Results of Standard Case (Hokusetsu Region)

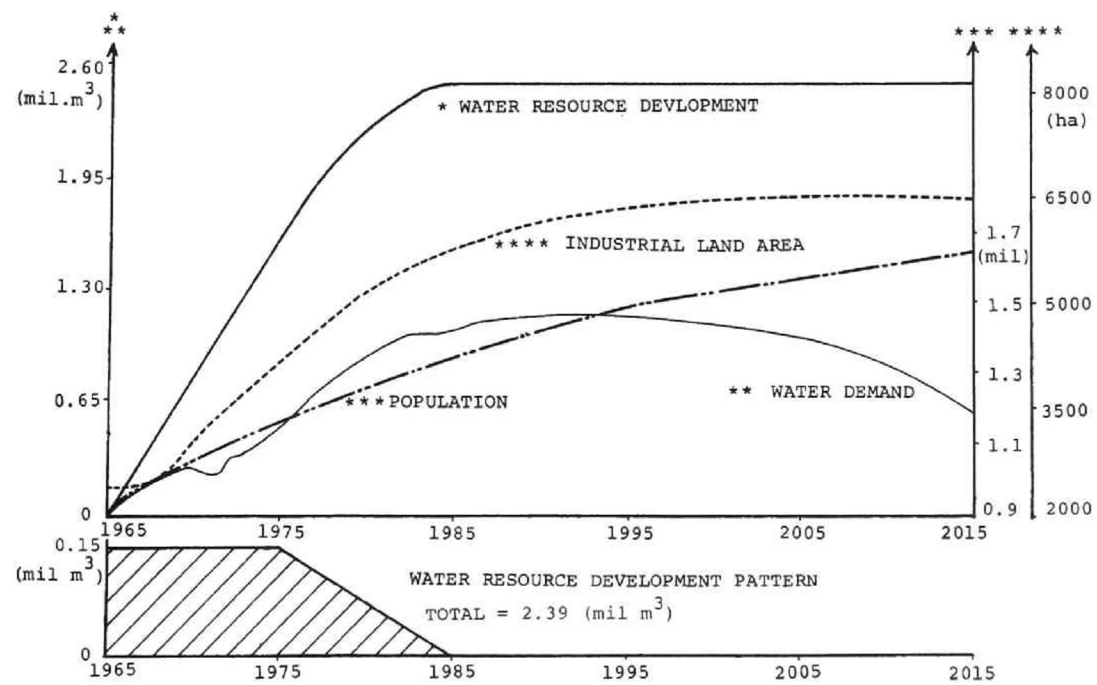


Fig. 2.5.5 Results of Standard Case (Tohban Region)

in the state a little bit favorable as compared to the Hokusetsu Region in connection to the water use management.

(iv) The occurrence of water shortages for a limited short time retards the growth rate of population to some extent.

2.5.3 Model Validation

In course of our research program, we stressed the use of sensitivity experiments as a means of identifying those parts of the model that are most sensitive and important. The results of these sensitivity experiments depend, however, on the structure of the model, and the conclusions drawn from those experiments are therefore based on the assumption that the model structure is a valid representation of the real world. The model's validity can be tested only by comparing its output to real world data. This may be done for example, by seeing how well the model can produce past regional performance. As is obvious in Table 2.5.3 there seems to be slight discrepancies in the simulation outputs and real world data. We concluded from this validation experiment that the model was behaving fairly well.

2.5.4 Comparison of Sensitivity Experiments

1) Population and Industrial Land Area Upper Limit Parameters

As easily expected from the difference in the upper limit parameter values for three different cases, population capacity at the end of the period gradually slowdowns from Cases I through III. This is also the case with land use area capacity. However a closer examination of the results shows that in such cases as Case T-I-B-1-a-α the population at the end of the period corresponds to its upper limit. By contrast in these cases land use area capacity falls short of its upper limit, while in Case T-I-A-a-α it falls short of its upper limit in the final stage. This suggests that the water supply development that is intensively promoted within the former half of the period leads to one of the upper limits. Stated otherwise, this kind of development accelerates the growth of the region, involving uncontrolled growths of both population and industrial land area, and consequently amounting to the capacity of the region.

NOTATION	CALUCULATED VALUE (I)	ACTUAL VALUE(II)	I/II
POPH	2,600,000	2,620,835	0.99
POPBH	52,300	53,190	0.98
POPCH	565,000	551,647	1.02
POPHH	213,000	209,065	1.02
POPYH	247,000	306,966	0.80
POPAH	502,000	507,866	0.99
POPMH	654,000	656,158	1.00
POPOH	363,000	335,943	1.08
AMSH	23,800	24,281	0.98
LFH	1,180,000	1,210,560	0.97
NEH	998,000	996,010	1.00
NEMH	259,000	257,293	1.01
AMH	2,270	2,245	1.01
PUAH	84.5	64.0	1.32
PUPH	0.319	0.316	1.01
IWDH1	936,000	735,342	1.27

Table 2.5.3 Validity Test

In other cases where water supply development works are maintained constantly over the entire period, or postponed until the latter half of the period, population and industrial land area generally fall below their capacities. Especially as the development is put off, the population at the end of the period tends to decrease even when we develop suppliable water in the latter half of the period on the same scale as in the former half.

2) Water Supply Development Patterns

As referred to in the above, other things being equal, the earlier the development is conducted, the more population the region can hold. But excessive amount of water that is developed in the incipient stage of the period will not necessarily lead to an increased population, because its further growth is restricted automatically owing to the population or industrial land limitation policies. In contrast to this if the development is postponed, resultant water shortages occurring frequently, retard the growth rate of regional activities, consequently leading to relatively undergrowth of the region at the end of the period.

In addition as illustrated by Figure 2.5.6 through 2.5.8, different water supply development patterns will result in those different paths of growths of the variables concerned, which are relatively similar to the path of water supply development.

3) Two Different Policy Patterns

Two kinds of policy parameters were set as shown in Figure 2.4.3. Table 2.5.4 shows the comparison of the outputs. From this we may conclude that difference in policy parameters produces relatively slight change in the outputs.

4) Two Different Water Demand Unit Projections

Figure 2.5.2 illustrates the two kinds of projections. A study of Table 2.5.5 shows that the results of many of our experiments seem to suggest that in the standard cases the drop in the industrial water demand unit tends to surpass the growth of population toward the end of the period, thus resulting in both the non-increasing water demand and excessive water supplies even if population and industrial land area are gradually getting closer to the upper limits.

By contrast in another case where industrial water demand unit is projected to take relatively higher (or ineffective) value toward the end of the period, the water demand and supply relation is sometimes well-balanced in the final stage, but in other cases the suppliable water falls short of the water demand, depending upon the water supply development patterns. From this we may conclude that if the standard water demand unit that is based on a rather optimistic expectation of industrial water use rationalization in the future, suppliable water may not necessarily come short of the water demand. This is, however, true only if the assumed amounts of suppliable water is practically feasible within the entire period. But this assumption seems to be too optimistic from the viewpoint of technological, economic and social situations. In practice construction of a dam needs more than a decade until it is completed and operation starts. Many of the social and economic problems involved in the planning and construction of a dam, are likely to double or sometimes triple the period needed

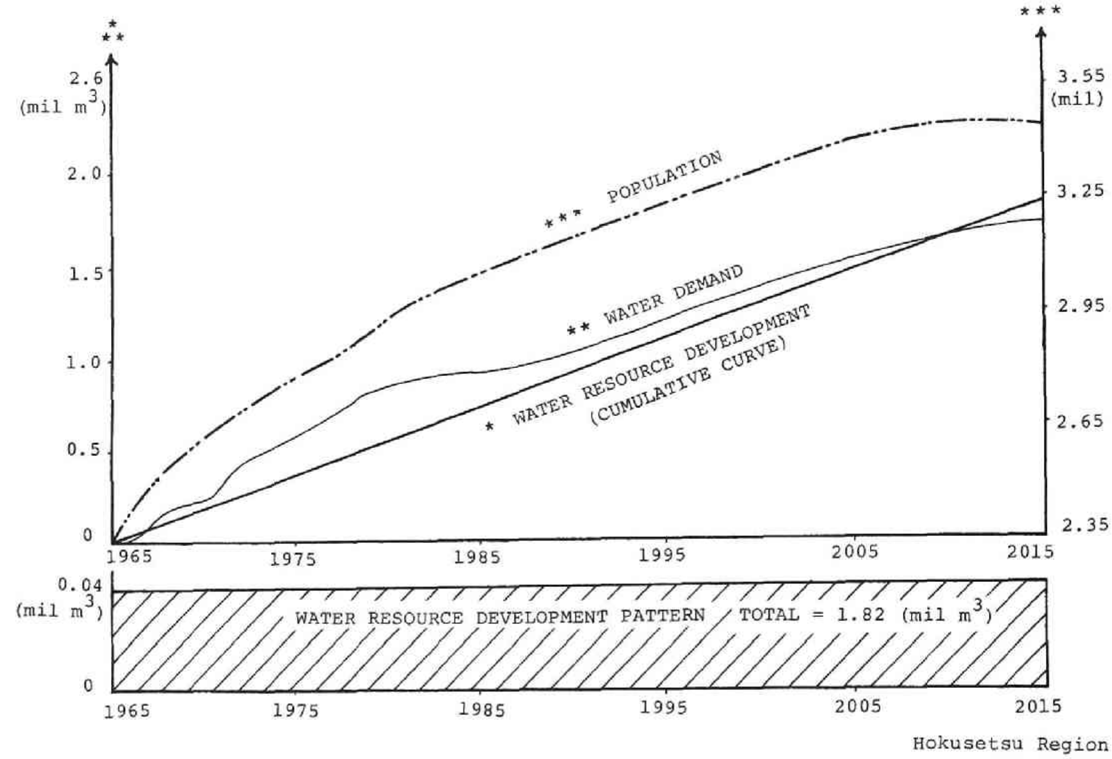


Fig. 2.5.6 Comparison of Outputs for Different Water Resources Development Patterns (1)

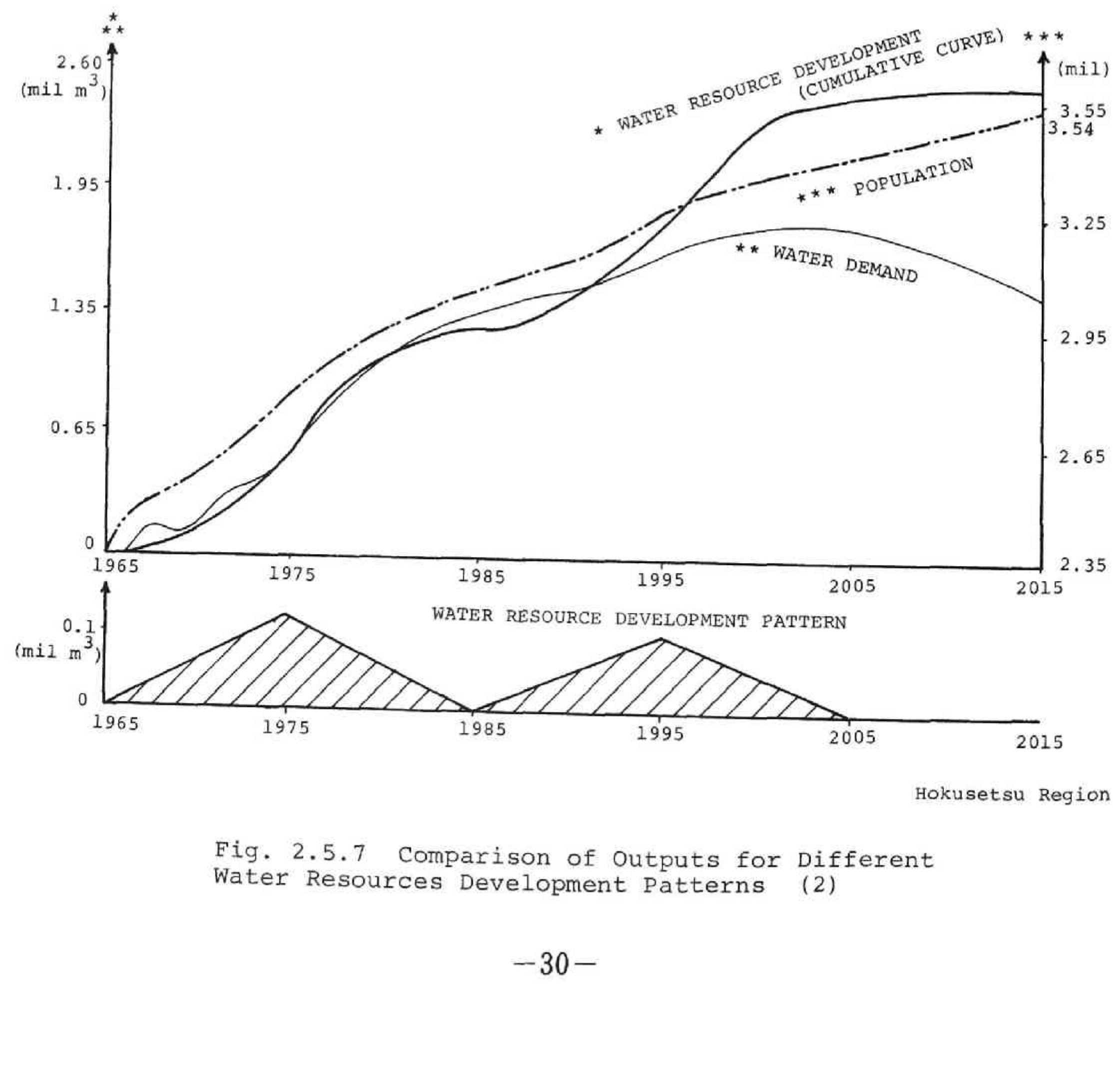


Fig. 2.5.7 Comparison of Outputs for Different Water Resources Development Patterns (2)

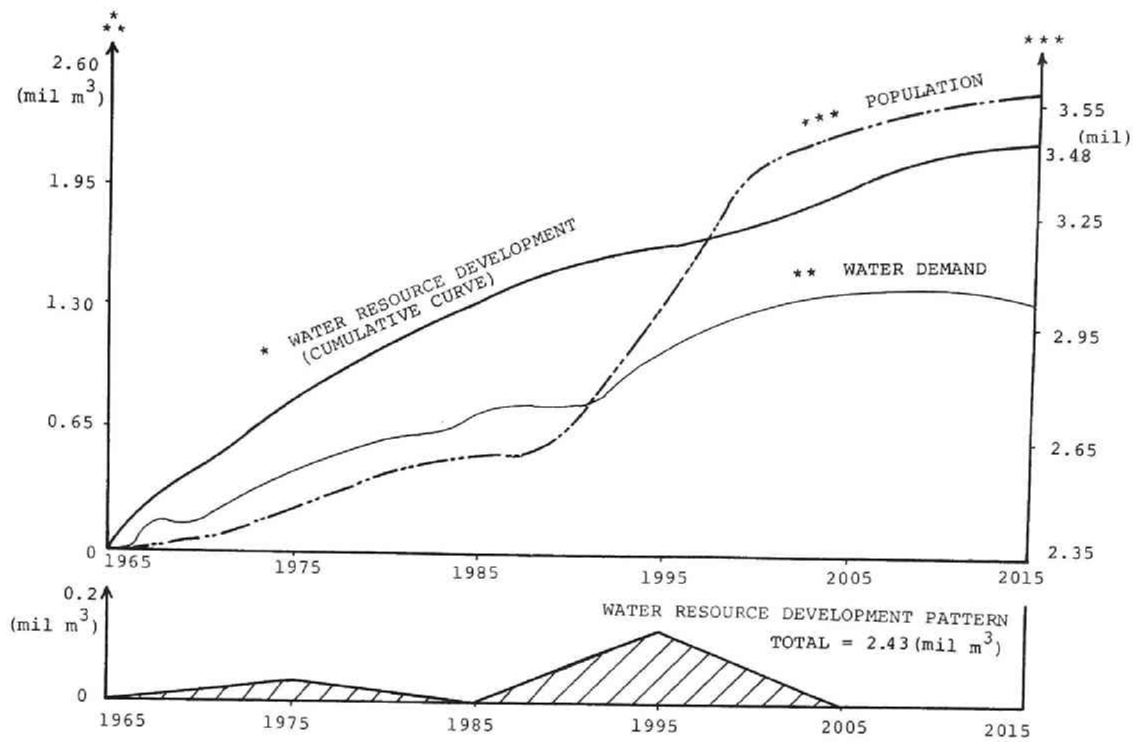


Fig. 2.5.8 Comparison of Outputs for Different Water Resources Development Patterns (3)

OUTPUT \ CASE	H-I-B-1-a-α	H-I-B-1-b-α
POPH	3.49 (mil)	3.48
AMH	2.763 (ha)	2.705
AMSH	0.117 (bil.yen)	0.103
PUPH	0.505 (m^3/capta)	0.500
PUAH	22 (m^3/mil.yen)	22
CWRH	0.763	0.763
WRH	0.929	0.929
WDT	1.30 (m^3)	1.28

Table 2.5.4 Comparison of Results for Different Development Patterns

OUTPUT \ CASE	H-I-A-a-α	H-I-A-a-β
POPH	3.65 (mil)	3.50
AMH	2.660 (ha)	2.620
AMSH	0.115 (bil.yen)	0.117
PUPH	0.581 (m^3/capta)	0.601
PUAH	22 (m^3/mil.yen)	41
CWRH	0.763	0.705
WRH	0.929	0.880
WDT	1.66 (m^3)	2.35

Table 2.5.5 Comparison of Results for Different Water Projection Unit Patterns

to complete the construction.

In this report it seems to be informative to compare the above cases with other cases where amounts of suppliable water for the supply to meet water demands are not developed continuously but stepwise at certain intervals. Figures 2.5.9 and 2.5.10 illustrate some of the typical examples of these cases. From this we see that water shortages do occur more frequently, though on a small scale, and the population and industrial land area of the regions concerned tend to fall far below their upper limits. Notably enough, the path of the growth of water demand fluctuates as suppliable water is developed step by step over the period.

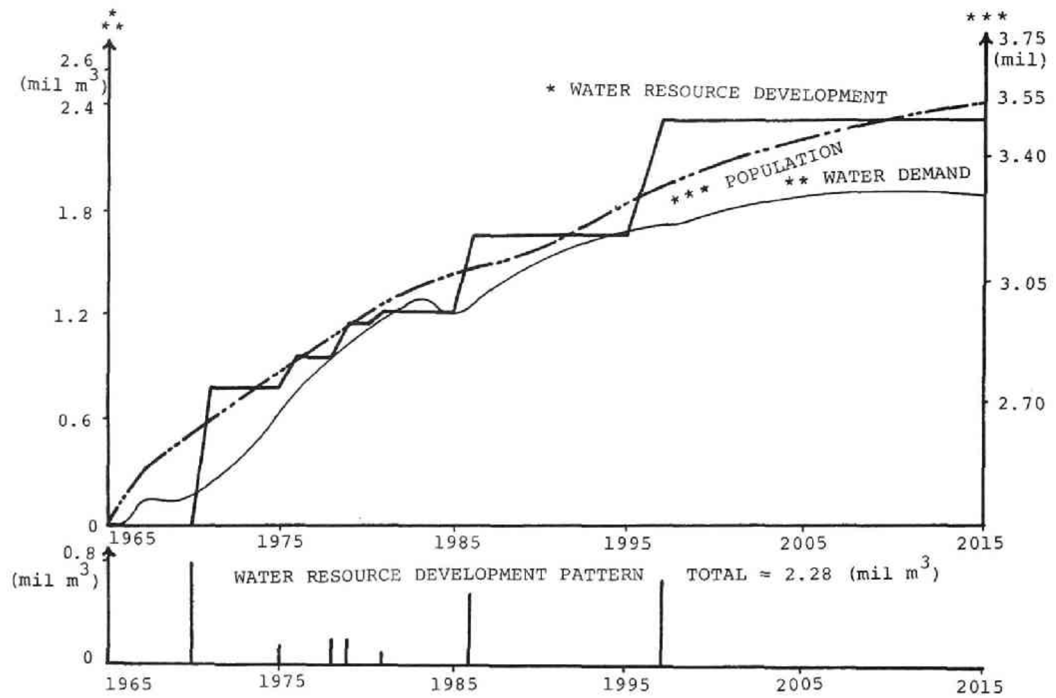


Fig. 2.5.9 Comparison of Outputs for a Stepwise Water Development Pattern (Hokusetsu Region, Case H-I-B-2-α)

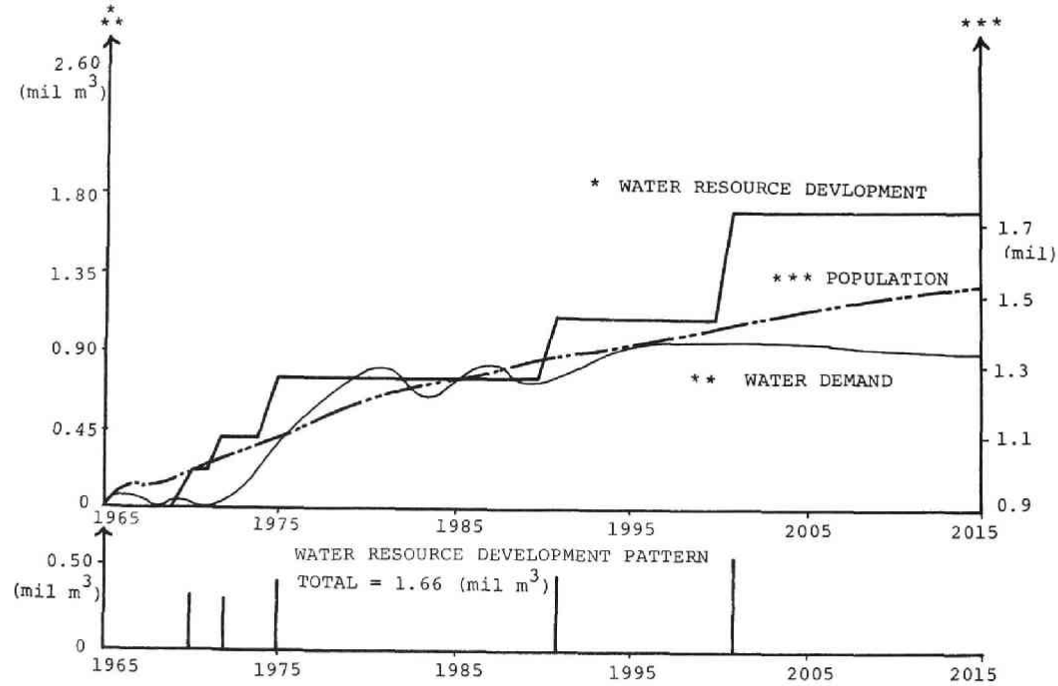


Fig. 2.5.10 Comparison of Outputs for a Stepwise Water Development Patter (Tohban Region, Case T-II-B-2-α)

2.6 Conclusion

A main cause for the syndrome of water scarcity may be attributed to the manner in which water resource management has conventionally been conducted in Japan. That is to say that water demands were estimated given prior assumptions about economic development patterns in the region and water resource development programs were implemented so as to meet those water demands to be produced in the projected regional activities. But if the economic development pattern is modified, water demands would be reduced. In other words the extent of water shortages is largely dependent on the assumed regional activity patterns, and in this sense water shortages should not necessarily mean the absolute stringency of water. Therefore it seems to be important that water resource management programs should be examined within the framework of regional development patterns which are set unfixed and remain changeable, where necessary.

This kind of arguments seem to gain more popularity over past several years as the high economic advancement has become less attractive to the public, living in highly urbanized regions with deteriorated environments and consequently as the environmental protection problems have become a major concern of the planner and public. But attention needs to be devoted to the point that reduced degree of economic activities, causing different effects on the complex of regional activities, would have much impact upon water resources and related activities, and subsequently the influenced water-concerned activities would have indirect impact backward upon the regional activities.

The present study represents an example of the use of simulation model to throw light on the question as summarized in the above. It is clearly the case that models like the present one, or the present one with suitable modification, can be of value in helping to appraise the possible effect of various kinds of water resource management policies on regional economic growth as well as its backward effects on the water-concerned policies. To make the point more clearly we now review briefly our simulation results.

(i)As far as the Tohban and Hokosetsu Regions are concerned, the water resource management policy seems to have much impact on the regional development, especially in the earlier stages of the period, say in 10 or 20 years, on the assumption that relatively efficient water use patterns coupled with optimistic water resource developments are hardly to be attained.

(ii) In most of the other cases where such kind of optimistic water resource developments are assumed to be possible, the population or industrial land area eventually reaches it upper limit, thereby resulting in the slowdown of the growth of the water demand.

(iii) These results seem to suggest that the development of the regions concerned will be, early or late, confronted with either of the two problems: shortages of water or land, any of which will restrict further growth of the regions.

We may conclude, therefore, that the management of water resources as well as land uses is an extremely important factor in planning the development of the regions. Such information could be of great value in reaching decisions with respect to the regional development, inclusive of the water resource development.

Finally we briefly explain how we think the model should be improved.

1. With our primary emphasis placed on the dynamic interaction between water-concerned and regional activities, we consciously simplified the regional structure, and in turn constructed rather complicate structure of water-concerned activities. For instance, the demographic activities are assumed to depend mainly on industrial activities. But in the real world they are interrelated to other factors such as commercial activities or public investment activities. It may be informative to introduce some kind of "attractiveness factor."
2. Two submodels are not directly joined with each other in conducting simulation runs. The introduction of these functions to intergrate the two submodels seems to deserve attention.
3. In relative to the water-concerned variables, pollution problems are not directly considered.

Furthermore in applying the model to regions other than those for which it was designed in this study, it may be necessary to introduce modifications, append additional sectors, and elaborate some sectors already in the model. Anyway we believe that the presented model coupled with specified examinations and modifications would become an effective tool in discussing the water resource management within the framework of the development problem in any region to be treated.

References

1) Ministry of Construction: Survey Report on Long-term Water Resource Management Perspectives, 1973 (In Japanese).
2) Okada, N., and Yoshikawa, K.: System Dynamics Approach to the Water Resources Management Related to the Regional Development, Proceedings Prepared for the 4th Pan Pacific Regional Conference, 1975.
3) Mochizuki, T.: Systems Analysis of Regional Water Resources Managements, Master Thesis Presented to the Faculty of Engineering, Kyoto University, March, 1975 (In Japanese).
4) Forrester, J.W.: Industrial Dynamics, The M.I.T. Press, 1961.
5) Forrester, J.W.: Urban Dynamics, The M.I.T. Press, 1969.
6) Forrester, J.W.: Industrial Dynamics; After the First Decade, Management Science, Vol.14, No.7, pp.398-415, March, 1968.
7) Hamilton, H.R., et al.: Systems Simulation for Regional Analysis; An Application to River Basin Planning, The M.I.T. Press, 1969.

Chapter 3 Nonlinear Programming Approach for the Analysis of Intra-basin, Multi-modal Water Utilization System

3.1 Introduction

In the preceding chapter we have concerned ourselves with the problem of finding effective alternatives for the water resources management on a regional basis and specific attention was given to the impacts of controlling the water requirements on the regional activites.

In this chapter assuming that the future development pattern of the regional economy is given a priori, attention is devoted to the problem of the development and utilization of an intra-basin (single-basin) water resources system.[1)2)3)] The major reasons for focussing on an intra-basin system are as follows.

(i) Since a river basin includes that portion of the earth's surface and subsurface which contribute to flow in a specific stream and because present-day water supplies owe a large portion of water to the water body, it can be conceived as a geographical unit for water supply sources as well as water uses.

(ii) Since water is collected from a given stream, used and partially discharged to it on the upperstream and then reused again on the downstream, the existing intra-basin systems are considered to be a kind of water resources recycling systems.

(iii) The intra-basin system can be thought of as a sub-system of the inter-basin development and utilization system which will specifically be discussed in the chapters that follow, i.e., Chapters 4 and 5.

Accordingly in prior to the analysis of inter-basin system, it seems quite natural to consider detailed aspects of intra-basin system. In accordance with the above discussion, we shall, first of all, try to specify the problem of this kind.

3.2 Identification of the Problem

(i) A specific single basin is considered where we assume that by taking account of the local differences in water usage, legislative and economical boundaries, etc., the river basin is a priori divided into a couple of subareas which will hereafter be referred to as "demand zones" or simply "zones".

(ii) In the headwaters of the stream the amount of fresh water tapped by dams are already determined a priori. It is assumed that the fresh water contributes to the augmentation of the streamflow, resulting in the increased availability of the amounts of water to be collected from it as well as the improved quality of the water body.

(iii) Each demand zone is considered a geographical unit for water utilization in which a dual-modal system, namely, a combined system for utilizing both fresh water and renovated water is assumed to be implemented.

(iv) There are two types of water users in each demand zone, i.e., industrial water and domestic water users.

(v) As shown in Figure 3.2.1, in each zone water is collected from the nearest stream running through the area, then undergoes purification at two different plants (one for industrial water supply and the other for domestic water supply) and is provided for both kinds of water users, from which come out wastewaters

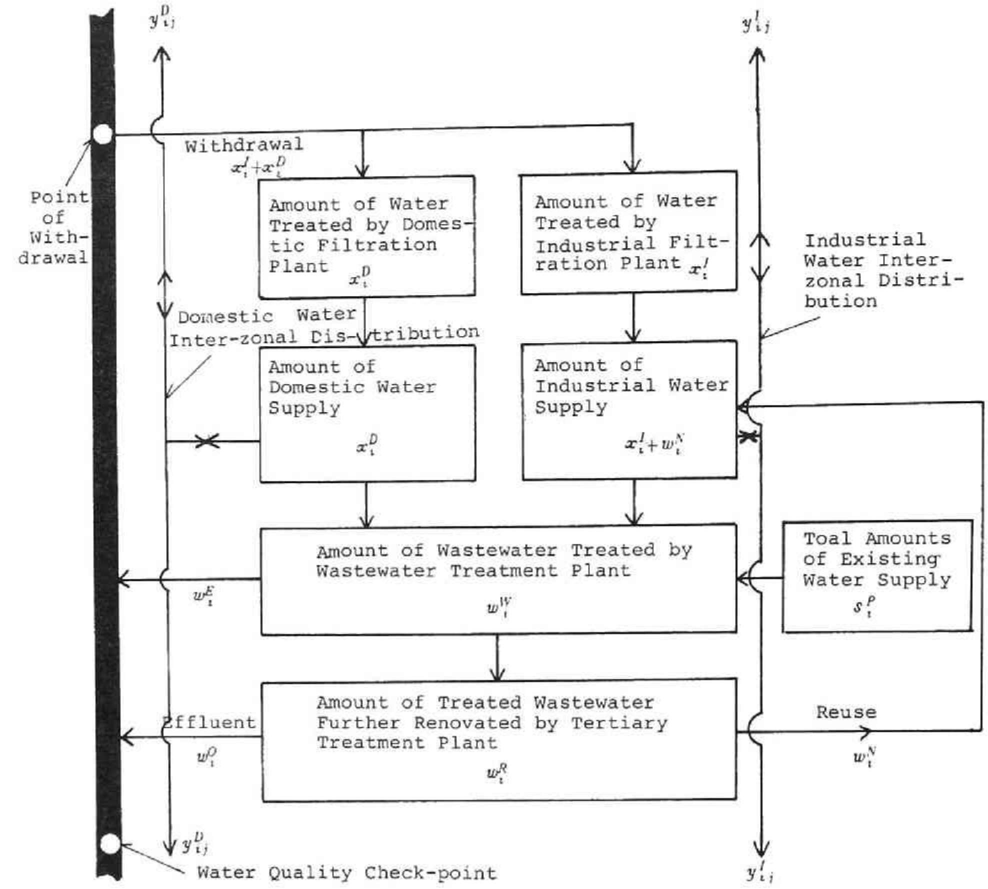


Fig. 3.2.1 Model Diagram

that are carried through the sewers to a wastewater treatment plant and that undergo the primary and secondary treatments to be partially returned back through an outlet to the same stream at a point of further downstream from the point of withdrawal, the remainder of which being further renovated by the tertiary process and then partially returned to the receiving water body through an outlet and partially provided again for the industrial water use.

(vi) Between a given water body and its adjacent ones is constructed diversion channels, if necessary.

(vii) To restate, the water utilization system of each zone is assumed to consist of those facilities for withdrawal (1), filtration (2), wastewater treatment (1), tertiary treatment (1), discharge (2) and distribution of renovated waters (1). The numbers in the parentheses represent the numbers of facilities involved.

(viii) In each zone at that point on the stream which is located most downstream from the point of withdrawal the water quality is required to meet the set standard.

(ix) The self-purification effects of the stream on the streamflow quality are considered to be negligible.

(x) The scale of water facilities concerned are determined so as to meet the incremental water demands which are predicted a priori.

(xi) The quality of the renovated water treated by the higher standard is considered to be roughly equal to that of the industrial water. The small portion of water demands for industrial uses is assumed to be covered by the domestic water supply.

(xii) The objective function is taken to be the minimized total costs for constructing above-cited facilities.

3.3 Model Formulation

3.3.1 Notation

1) Variables

x_i^I : amount of water treated by the filtration plant for industrial water supply to be constructed in zone i

x_i^D : that for domestic water supply

y_{ij}^I: amount of purified industrial water to be distributed from zone i to zone j

y_{ij}^D: that of purified domestic water

w_i^W: amount of wastewater treated by the wastewater treatment plant to be constructed in zone i

w_i^E : amount of treated wastewater to be discharged from zone i into the receiving water body

w_i^R : amount of treated wastewater to be further renovated by the tertiary plant

w_i^N: amount of renovated water to be reused by industry

w_i^Q : amount of renovated water to be discharged into the receiving water body

Q_i^L : average annual discharge at the downstream-most point in zone i

Q_i^U : that at the upperstream-most point

B_i^L: average BOD at the downstream-most point (check-point) in zone i

B_i^U: that at the upperstream-most point

2) Constants

w_l : amount of fresh water to be tapped by dam l constructed in the headwaters

q_i^1 : minimum discharge requirement of that part of the stream running through zone i

q_i^2: amount of streamflow currently being collected by existing water users in zone i

q_i^3 : inflow from those tributary streams joining the main stream at the points located between further downstream from the withdrawal point of the nearest upperstream zone to zone i and further upperstream from the withdrawal point of the zone i

D_i^I : water demand for industrial uses

D_i^D: that for domestic uses

s_i^P: total amount of existing water supply for industrial and domestic uses in zone i

b_i^3: average BOD of inflow from the tributary streams

b^E: average BOD of effluents from the wastewater treatment plant (same value for each zone)

b^O: average BOD of renovated waters (same value for each zone)

$\overline{B}_l$: BOD standard prescribed a priori at the check-point downstream-most of the stream

α_i^C : unit cost associated with construction and maintenance of the collection facilities in zone i

α_i^I : that of the industrial water filtration facilities in zone i

α_i^D : that of the domestic water filtration facilities in zone i

$\beta_{ij}^I (\beta_{ji}^I)$: that of the industrial-water distribution aqueduct which conveys water from zone(i) to (j)

β_{ij}^D : that of the domestic water distribution aqueduct

γ_i^W : that of the wastewater treatment plant

γ_i^R : that of the tertiary treatment plant

γ_i^E : that of the wastewater effluent facilities

γ_i^Q : that of the renovated water effluent facilities

γ_i^N : that of the renovated water distribution conduits

δ_l : that of dam l

3.3.2 Constraints

In addition to the nonnegativity conditions of all the variables, we set the following constraints for each zone $i=1,\cdots\cdots,m$.

Available amount of streamflow to be collected is constrained as

$$x_i^I + x_i^D \leq Q_i^U - q_i^1 - q_i^2 \quad \cdots\cdots (3.3.1)$$

For industrial supply of water it follows

$$D_i^I \leq x_i^I + \sum_{j=i^* \text{or } i^{**}} (y_{ji}^I - y_{ij}^I) + w_i^N \quad (i=1,\cdots\cdots,m) \quad \cdots\cdots (3.3.2)$$

Likewise for domestic supply of water

$$D_i^D \leq x_i^D + \sum_{j=i^* \text{or } i^{**}} (y_{ji}^D - y_{ij}^D) \quad (i=1,\cdots\cdots,m) \quad \cdots\cdots (3.3.3)$$

where the subscripts i^* and i^{**} are used to refer to those zones located nearest upward or downward from zone i.

The amount of wastewater treated by the wastewater treatment plant in zone i is so limited

$$w_i^W \leq D_i^I + D_i^D + s_i^P \quad (i=1,\cdots\cdots,m) \quad \cdots\cdots (3.3.4)$$

And the amount of renovated water treated by the tertiary plant in zone i and that of renovated water to be reused in this zone are so constructed

$$w_i^R = w_i^W - w_i^E \quad (i=1,\cdots\cdots,m) \quad \cdots\cdots (3.3.5)$$

$$w_i^N = w_i^R - w_i^Q \quad (i=1,\cdots\cdots,m) \quad \cdots\cdots (3.3.6)$$

The average annual discharge of the stream at the withdrawal point most upperstream in zone i is governed by the streamflow condition of the nearest upward zone as follows.

$$Q_i^U = \sum_{i' \in U_i} q_{i'}^3 + \sum_{i' \in U_i} (w_{i'}^E + w_{i'}^Q) - \sum_{i' \in U_i} (x_{i'}^I + x_{i'}^D) + \sum_l u_l \quad (i=1,\cdots\cdots,m) \quad \cdots\cdots (3.3.7)$$

where U_i denotes the set of those zones located further upperstream from zone i.

Substitution of Equation (3.3.7) into (3.3.1) yields

$$x_i^I + x_i^D + \sum_{i' \in U_i} (x_{i'}^I + x_{i'}^D) \leq \sum_{i' \in U_i} q_{i'}^3 - q_i^1 - q_i^2 + \sum_{i' \in U_i} (w_{i'}^E + w_{i'}^O) + \sum_{l=1}^{L} w_l \quad \text{(3.3.8)}$$

The water quality of the stream at the check-point downstream-most in zone i is confined to meet the set standard. This is formulated as follows.

$$B_i^L \leq \overline{B}_i \quad (i=1, \cdots, m) \quad \text{(3.3.9)}$$

where

$$B_i^L = \frac{1}{Q_i^L}\{ B_i^U (Q_i^U - x_i^I - x_i^D) + b_i^3 q_i^3 + b^E w_i^E + b^O w_i^O \} \quad (i=1, \cdots, m) \quad \text{(3.3.10)}$$

Let us also assume here

$$Q_i^U = Q_{i*}^L \quad (i=1, \cdots, m) \quad \text{(3.3.11)}$$

$$B_i^U = B_{i*}^L \quad (i=1, \cdots, m) \quad \text{(3.3.12)}$$

By substituting Equations (3.3.10), (3.3.11), (3.3.12) into (3.3.9), we obtain the following recurrence relations with respect to each zone.

$$\frac{1}{Q_i^L}\{ B_{i*}^L (Q_{i*}^L - x_i^I - x_i^D) + b_i^3 q_i^3 + b^E w_i^E + b^O w_i^O \} \leq \overline{B}_i \quad \text{(3.3.13)}$$

Since B_{i*}^L is also required to meet Equation (3.3.10) (i being replaced by i^*), we know that Equation (3.3.13) is a nonlinear inequality.

3.3.3 Objective Function

The objective function is taken to be the minimization of the total costs, which is formulated as follows.

$$\begin{aligned} \text{Minimize} \quad Z = \sum_i \{ & \alpha_i^I x_i^I + \alpha_i^D x_i^D + \alpha_i^C (x_i^I + x_i^D) \\ & + \sum_{j=i^*, i^{**}} (\beta_{ij}^I y_{ij}^I + \beta_{ji}^I y_{ji}^I + \beta_{ij}^D y_{ij}^D + \beta_{ji}^D y_{ji}^D) \\ & + \gamma_i^W w_i^W + \gamma_i^R w_i^R + \gamma_i^E w_i^E + \gamma_i^O w_i^O + \gamma_i^N w_i^N \} + \sum_l \delta_l w_l \end{aligned} \quad \text{(3.3.14)}$$

To sum up the above formulations of the model, they are rewritten as follows.

3.3.4 Formulated Model

[Objective Function]

$$\begin{aligned} \text{Minimize} \quad Z = \sum_i \{ & \alpha_i^I x_i^I + \alpha_i^D x_i^D + \alpha_i^C (x_i^I + x_i^D) \\ & + \sum_{j=i^*, i^{**}} (\beta_{ij}^I y_{ij}^I + \beta_{ji}^I y_{ji}^I + \beta_{ij}^D y_{ij}^D + \beta_{ji}^D y_{ji}^D) \\ & + \gamma_i^W w_i^W + \gamma_i^R w_i^R + \gamma_i^E w_i^E + \gamma_i^O w_i^O + \gamma_i^N w_i^N \} + \sum_l \delta_l w_l \end{aligned} \quad \text{(3.3.14)}$$

[Linear Constraints]

$$x_i^I + x_i^D \leq Q_i^U - q_i^1 - q_i^2 \quad (i=1, \cdots, m) \quad \text{(3.3.1)}$$

$$D_i^I \leq x_i^I + \sum_{j=i^* \text{or } i^{**}} (y_{ji}^I - y_{ij}^I) + w_i^N \quad (i=1,\cdots\cdots,m) \qquad (3.3.2)$$

$$D_i^D \leq x_i^D + \sum_{j=i^* \text{or } i^{**}} (y_{ji}^D - y_{ij}^D) \quad (i=1,\cdots\cdots,m) \qquad (3.3.3)$$

$$w_i^W \geq D_i^I + D_i^D + s_i^P \quad (i=1,\cdots\cdots,m) \qquad (3.3.4)$$

$$w_i^R = w_i^W - w_i^E \quad (i=1,\cdots\cdots,m) \qquad (3.3.5)$$

$$w_i^N = w_i^R - w_i^O \quad (i=1,\cdots\cdots,m) \qquad (3.3.6)$$

$$Q_i^U = \sum_{i' \in U_i} q_{i'}^3 + \sum_{i' \in U_i} (w_{i'}^E + w_{i'}^O) - \sum_{i' \in U_i} (x_{i'}^I + x_{i'}^D) + \sum_l w_l \quad (i=1,\cdots\cdots,m) \qquad (3.3.7)$$

$$Q_i^U = Q_{i*}^L \quad (i=1,\cdots\cdots,m) \qquad (3.3.11)$$

$$B_i^U = B_{i*}^L \quad (i=1,\cdots\cdots,m) \qquad (3.3.12)$$

[Nonlinear Constraints]

$$B_i^L = \frac{1}{Q_i^L} \{ B_{i*}^L (Q_{i*}^L - x_i^I - x_i^D) + b_i^3 q_i^3 + b^E w_i^E + b^O w_i^O \} \leq \overline{B}_i \quad (i=1,\cdots\cdots,m) \qquad (3.3.13)$$

3.4 Solution Technique

3.4.1 Preliminary Discussion

The above formulated model is a nonlinear programming with a specific property that a grater part of the constraints are linear.

One straightforward way to handle this type of problem is to incorporate all the constraints, whether nonlinear or linear, into a modified objective function by use of penalty factors. Many approaches have recently been developed on the basis of this idea. We shall utilize the method of Fiacco and McCormic[4)5)6)] but some modification will be made in the light of the following considerations.

(i) Many computational experiences have shown that the penalty method works well only when the number of those constraints to be incorporated into the objective function is relatively small, whereas in case the number is large, it is found to be inefficient and there is no guarantee of convergence to the desired optimum within a predetermined accuracy. Accordingly some algorithmic device is needed to reduce the number of those constraints to be incorporated into the objective function.

(ii) The special type of problem, i.e., that of finding a minimum (or maximum) for a linear set of constraints has proven to be very effectively approachable by those variants of gradient methods. Methods include that of feasible directions due to Zoutendijk[7)], those of gradient projection method developed by Rosen[8)9)], the cutting plane method of Kelley[10)11)] and so on[12)13)14)].

The above considerations have led the authors to the idea of combining the two kinds of techniques, that is, both the method of penalty function by Fiacco and McCormic and that of feasible directions due to Zoutendijk. That is to say that the original nonlinear programming problem is rendered to that special type of nonlinear programming with a property that only the objective function is nonlinear which is confined by a set of linear constraints. More detailed description of the algorithm will immediately follow.

3.4.2 Developed Algorithm

Let the problem be defined as follows.

$$\text{Minimize } Z=f(\boldsymbol{x}) \quad \cdots\cdots (3.4.1)$$

$$A\boldsymbol{x} \geqq \boldsymbol{b} \quad \cdots\cdots (3.4.2)$$

$$g_i(\boldsymbol{x}) \geqq 0 \quad (i=1, \cdots\cdots, m) \quad \cdots\cdots (3.4.3)$$

$$\boldsymbol{x} \geqq 0 \quad \cdots\cdots (3.4.4)$$

where $f(\boldsymbol{x})$ and $g_i(\boldsymbol{x})$ represent nonlinear functions.

step 1

Find anyhow an arbitrary feasible solution $\boldsymbol{x}^{(1)}$ to the above nonlinear problem. As a matter of fact this step should be backed up by some sophisticated algorithm for finding an initial feasible solution. We have developed an effective sub-algorithm for this purpose whose explanations will be deferred to 3.4.3 in order to rough out the major algorithm beforehand.

step 2

For a properly predetermined penalty factor $p^{(K)}$ (K denoting the number of iterations of the process being discussed now).

Let us convert the above nonlinear programming problem into that type of nonlinear problem of optimizing a nonlinear objective function with a linear set of constraints.

$$\text{Minimize } F^{(K)}= Z-p^{(K)}\sum_{i=1}^{m}\frac{1}{g_i} \quad \cdots\cdots (3.4.5)$$

subject to

$$A\boldsymbol{x} \geqq \boldsymbol{b} \quad \cdots\cdots (3.4.2)$$

$$\boldsymbol{x} \geqq 0 \quad \cdots\cdots (3.4.4)$$

The above formulated model will be handled by another sub-algorithm based on Method of Zoutendijk whose explanation will be given later in 3.4.4. Putting aside the detailed explanation of it for the moment, we shall proceed to the remaining part of the major algorithm.

By starting from the initial base point $\boldsymbol{x}^{(K)}$ which has been obtained beforehand, solve the above nonlinear programming by the method of Zoutendijk. Then we take the optimum of this problem $\boldsymbol{x}_o^{(K)}$ as our next initial base point $\boldsymbol{x}^{(K+1)}$.

At the first iteration $(K=1)$, we skip step 4 to follow, and proceed to step 5. Otherwise we go to step 4.

step 4

Compare the old optimum of the penalty function $F^{(K)}$ for the K-th iteration with the new one of $F^{(K+1)}$ for the $K+1$-th iteration and check whether the following condition holds.

$$\left|F^{(K+1)}-F^{(K)}\right| < \varepsilon \quad \cdots\cdots (3.4.6)$$

where ε is a predetermined tolerance for the convergence of the algorithm.

If the above condition holds, the above procedure terminates and we shall take this optimum as our desired one. Otherwise we step up to step 5.

step 5

Reset the value of $p^{(K)}$ as

$$p^{(K+1)}=\frac{p^{(K)}}{M^{(K)}} \quad \cdots\cdots (3.4.7)$$

where $M^{(K)}$ is the modification factor of $p^{(K)}$ for the K-th iteration to give a series of $p^{(1)} > p^{(2)} > \cdots\cdots > p^{(K)} > p^{(K+1)} > \cdots\cdots$ which converge to zero.

Then we return back to step 2 and the procedures should be repeated between steps 2 and 5 until the above convergence condition is found to be satisfied.

Two points seem to be needed to receive further discussions at a greater length.

One is the explanation of the sub-algorithm based on the method of Zoutendijk and the other the development of some efficient technique for finding a feasible solution. The remainder of this section (3.4.3 and 3.4.4) will be devoted to the discussions of these two algorithms.

3.4.3 Techniques for Predetermining Initial Base Point (Sub-algorithm 1)

As explained before step 1 of the major algorithm needs to be based on some sophisticated sub-algorithm for predetermining the initial base point.

One straightfoward way to set an initial base point is to find any feasible solution to the model, whether derived from the computation by hand or on digital computer. One fatal drawback to this method is the inefficiency involved in the selection of an arbitrary feasible point with no account taken of the evaluation of the objective function.

In light of this considration an attempt will be made to present an efficient technique for predetermining some appropriate base point which will lead to the exact solution (or its precise approximation) within a limited number of iterations. In this regard it should be noted that the nonlinear inequalities of (3.3.9) can be rendered to the mold of linearity by replacing the variables B_i^L and $B_i^U (= B_{i*}^L)$ by $\overline{B}_i$ and $\overline{B}_{i*}$, respectively. That is,

$$\overline{B}_i = \frac{1}{Q_i^L} \{ \overline{B}_{i*} (Q_{i*}^U - x_i^I - x_i^D) + b_i^3 q_i^3 + b^E w_i^E + b^O w_i^O \} \quad \cdots\cdots (3.4.8)$$

The above modification implies that in each zone the water quality control is required to be performed, so as to meet the most pessimistic case that the quality of the river flow running down to the zone in question from its nearest upstream zone happens to be equal to the quality limit, namely the standard BOD $\overline{B}_i$. In consequence the optimal solution to this linearized-constraint model is expected to be a good candidate for the initial base point.

It is inferred that this will be especially true in case relatively higher standards are imposed on the water quality control.

In accordance with the anove discussion we shall utilize the modified model as a tool for finding an initial base point and compare the results thus obtained with those derived from that search starting from an arbitrarily-chosen point.

3.4.4 Method of Feasible Directions (Sub-algorithm 2)

The method of Zoutendijk called feasible directions is used to handle the following type of nonlinear programming problem.

$$\text{Minimize} \quad W = F(\boldsymbol{x}) \quad \cdots\cdots (3.4.9)$$

subject to

$$A\boldsymbol{x} \geqq \boldsymbol{b} \quad \cdots\cdots (3.4.10\text{-}1)$$

$$\text{or} \quad \sum_{j=1}^{n} a_{ij} x_j - b_i \geqq 0 \quad (i = 1, \cdots\cdots, m) \quad \cdots\cdots (3.4.10\text{-}2)$$

where $F(x)$ is a nonlinear objective function with respect to $x(x_1, \cdots\cdots, x_n)$.

This method moves, essentialy, in the search for a maximum, until a boundary is encountered. When the search is made within the feasible region but not exactly on the boundaries, the gradient of the objective function determines the direction of movement as

$$t_j = \frac{-W_{x_j}}{\sum_{j=1}^{n} \left| W_{x_j} \right|} = \frac{-\dfrac{\partial F}{\partial x_j}}{\sum_{j=1}^{n} \left| \dfrac{\partial F}{\partial x_j} \right|} \quad \cdots\cdots (3.4.11)$$

Once the direction of movement is found, a one-dimensional search is conducted along the direction and an optimum found, if one exists. The search stops, however, when constraint boundaries are encountered. We therefore need to determine which is the smaller, the distance measured from the base point to the optimum along the line of search or the distance to the first violated constraint. Since the point $x^{(k)}$ is feasible (k denoting the number of iterations for the search by the method of Zoutendijk), we know that

$$\sum_{j=1}^{n} a_{ij} x_j^{(k)} - b_i \geqq 0 \quad (i = 1, \cdots\cdots, m) \quad \cdots\cdots (3.4.12)$$

In moving from $x^{(k)}$ along the direction of search, we have

$$\sum_{j=1}^{n} a_{ij} x_j^{(k)} - b_i + h \sum_{j=1}^{n} a_{ij} t_j^{(k)} \geqq 0 \quad \cdots\cdots (3.4.13)$$

where h denotes the distance of movement.

Which of these m constraints are in danger of being violated as h increases? Certainly, only those for which $\sum_{j=1}^{n} a_{ij} t_j^{(k)} < 0$ can possibly be violated.

Assume that there are $r (r \leqq m)$ such constraints. The greatest allowable distance of movement h_i without violating a constraint, is found by solving the equation

$$h_i = \frac{b_i - \sum_{j=1}^{n} a_{ij} x_j^{(k)}}{\sum_{j=1}^{n} a_{ij} t_j^{(k)}} \quad \cdots\cdots (3.4.14)$$

for the r inequalities for which $\sum_{j=1}^{n} a_{ij} t_j^{(k)} < 0$ holds.

The distance of movement is selected as

$$h = \min (h^*, h_{m_1}, h_{m_2}, \cdots, h_{m_r}) \quad \cdots\cdots (3.4.15)$$

Here h^* is the distance to the optimum along the line of search. A new base point is thus located.

If $h = h^*$, then the procedure is repeated. If $h = h_{m_0}$ (m_0 denoting one or more than one of the subscripts $1, \cdots\cdots, m$), we are on a boundary (boundaries), since

$$\sum_{j=1}^{n} a_{ij} x_j^{(k+1)} - b_i = 0 \quad \text{for} \quad i = m_0 \quad \cdots\cdots (3.4.16)$$

The constraints of this kind are called "operative constraints". We must therefore select the new direction of movement so as to satisfy the operative constraints. The movement will hopefully, be small enough so that the other constraints will not be violated. All the constraints must therefore be checked before finally adopting the new point. For the operative constraints we require that

$$\sum_{j=1}^{n} a_{ij}\, t_j^{(k+1)} \geq 0 \quad \cdots\cdots (3.4.17)$$

In addition we shall require that the $t_j^{(k+1)}$ shall satisfy the linear restriction.

$$-1 \leq t_j^{(k+1)} \leq 1 \quad \cdots\cdots (3.4.18)$$

This equation permits us to control the distance of movement by selecting values of h, since the $t_j^{(k+1)}$ are forced to remain prescribed limits. We now wish to select the $t_j^{(k+1)}$ so as to optimize the ΔW as defined in the below, while satisfying the inequalities (3.4.17) and (3.4.18)

$$\Delta W = h \sum_{j=1}^{n} W_{x_i}\, t_j^{(k+1)} = h \sum_{j=1}^{n} t_j^{(k+1)} \frac{\partial F}{\partial x_i} \quad \cdots\cdots (3.4.19)$$

This is a linear problem which can be solved using the simplex method of linear programming, provided we make the following transformation:

$$t_j^{(k+1)} = e_j^{(k+1)} - 1 \quad \cdots\cdots (3.4.20)$$

Here the new variable $e_j^{(k+1)}$ have the property of being nonnegative as contrasted with the $t_j^{(k+1)}$, which can be negative.

Substituting Equations (3.4.20) into (3.4.16), (3.4.17) and (3.4.18), our problem can be redefined as the selection of the n variables of $e_j^{(k+1)}$ which optimize

$$\sum_{j=1}^{n} W_{x_i}(e_j^{(k+1)} - 1) \quad \cdots\cdots (3.4.21)$$

subject to the constraints

$$\sum_{j=1}^{n} a_{ij}\, e_j^{(k+1)} - \sum_{j=1}^{n} a_{ij} \geq 0 \quad \cdots\cdots (3.4.22)$$

and $\quad e_j^{(k+1)} - 2 \leq 0 \ (j=1,\cdots\cdots,n) \quad \cdots\cdots (3.4.23)$

for $\quad e_j^{(k+1)} \geq 0 \ \ (j=1,\cdots\cdots,n) \quad \cdots\cdots (3.4.24)$

The solution of this linear problem defines a new direction of search which will satisfy the operative constraints. The new point found by moving a distance h along this direction should be tested to see, if the gradient vanishes at an interior point or if the $t_j^{(k+1)}$ vanishes at a boundary. Such being the case, a local optimum has been found and the procedure terminates. Otherwise the above iterative procedure is repeated.

3.5 Case Study on the Basin of the Kakogawa River

3.5.1 Regional Setting

We shall consider the basin of the Kakogawa River as the case study area. The Kakogawa River drains the most urbanized and industialized region of Hyogo Prefecture covering 21 percent of the total area, into the Seto Inland Sea. The drainage area includes Kakogawa and Takasago Cities where growing industrial activities are requiring extended developments of water resources on one hand and polluted stream waters are becoming a serious matter on the other hand.

In view of this situation we selected this basin as our case study area.

3.5.2 Model Data

1) Zoning

Taking account of local differences in hydrology and water uses, as well as legislative and economical boundaries, the study area was divided into three zones. (See Figure 3.5.1.)

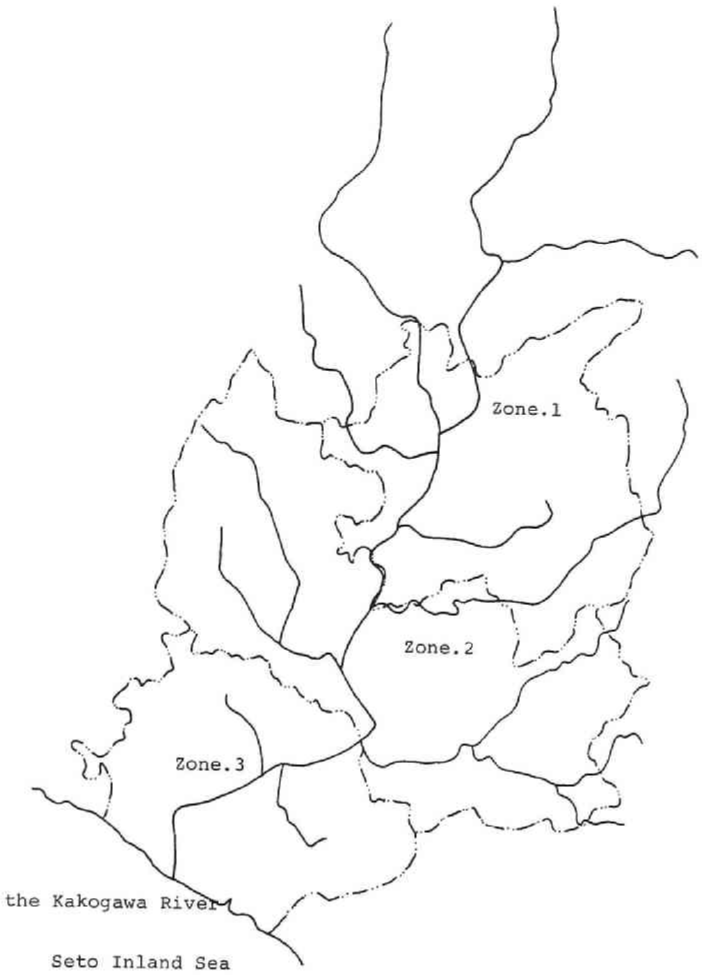


Fig. 3.5.1 Zoning of Study Area

2) Water Demands

By setting each zone as the geographical unit for predicting the water demands and forecasting the per-capita demands, per-unit-production demands at the time of 1985 and the increased population and production by that time, the predicted water demands were estimated as tabulated in Table 3.5.1.

3) River Discharges

By constructing the flow-duration curve for each river based on the data provided by the authorities concerned, that value of flow was obtained such that the percent of time it is equal to or less than that value is around 20 percent. We set the average of each of the preceding five years (1970-1974) as the sum of the minimal discharge requirements q_i^1 and the amounts of existing collected waters q_i^2.

As for the available fresh waters to be developed by dams, they are estimated as 0.5 million m^3/day on the basis of the materials provided by the Ministry of Construction.[15] This value was set as the standard value to be used for the standard case as will be explained later.

Zone \ Demand	Industrial Water Demand* ($10^3\ m^3$/day)	Domestic Water Demand* ($10^3\ m^3$/day)
Zone 1	26.9	10.0
Zone 2	84.5	16.7
Zone 3	182.2	60.6

* Increased Water Demand (1974-85)

Table 3.5.1 Projected Water Demands

4) Estimated Cost Curves

Practical experience shows that many, if not most, of the cost curves in question exhibit the mold of linearity. Some exceptions to this are those cost curves for the filtration plant, tertiary treatment plant and distribution conduit which proved to be expressed in the form of nonlinear concave functions as shown in Figures 3.5.2, 3.5.3 and 3.5.4.

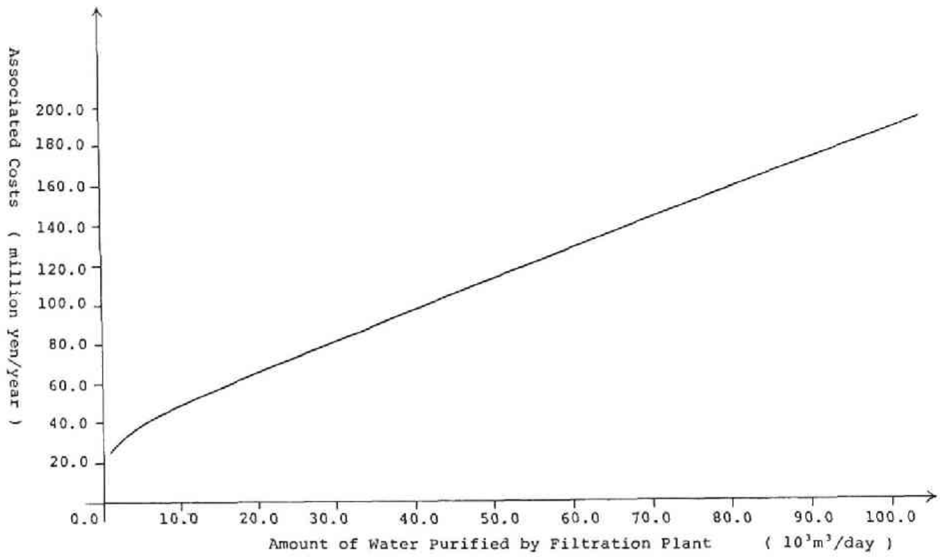


Fig. 3.5.2 Cost Curve for Filtration Plant

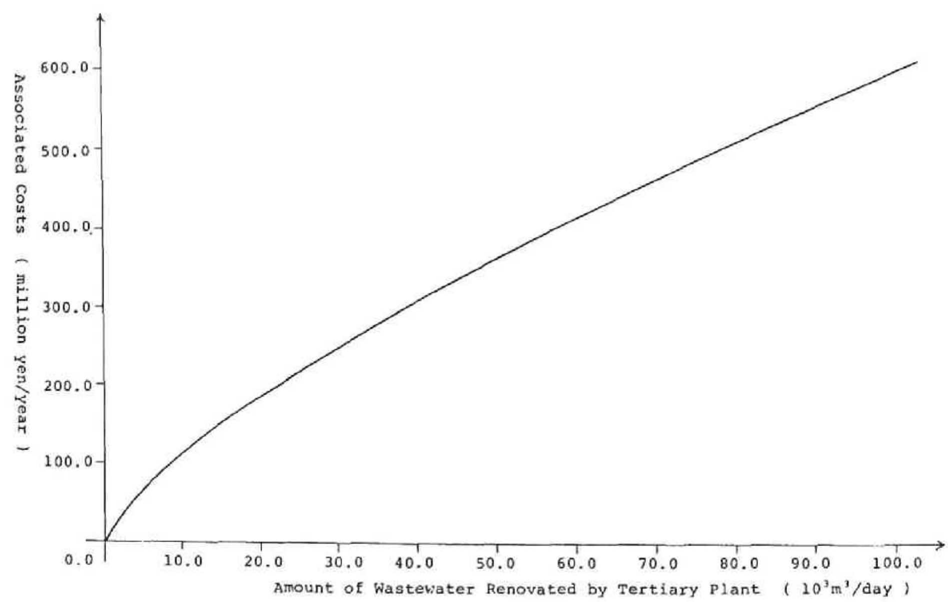


Fig. 3.5.3 Cost Curve for Tertiary Treatment

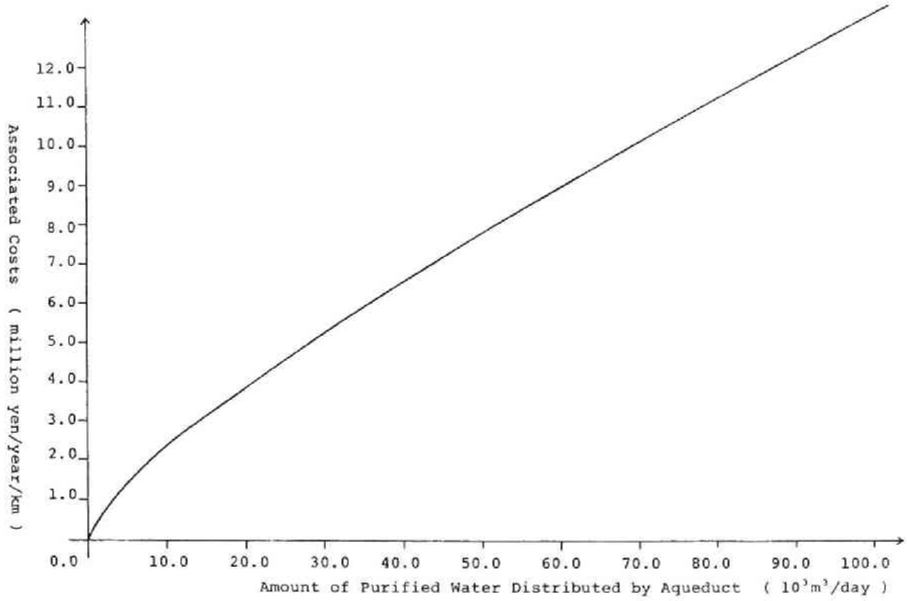


Fig. 3.5.4 Cost Curve for Distribution Conduit

5) Water Quality Values

The quality of the wastewater treated by the wastewater treatment plant was set to be 20 ppm by considering the values commonly experienced in the existing plants. The quality of the renovated wastewaters was taken to be 4 ppm, assuming that the higher standard processes are employed for the tertiary treatment as shown in Figure 3.5.5. The quality of every tributary stream is equally assumed to be 2 ppm, providing that it is constrained to satisfy the standard value as legislatively prescribed by the authority concerned.[16)17)] The set quality of the most upperstream of each river is 3 ppm.

As for the quality of different parts of the streams, it was set as follows.

① $\bar{B}_0=3.0$, $\bar{B}_1=5.0$, $\bar{B}_2=7.0$, $\bar{B}_3=8.5$, $\bar{B}_4=10.0$ (ppm)
($\bar{B}_0= B^L_{1*} = B^U_1$)
② $\bar{B}_0=3.0$, $\bar{B}_1=4.0$, $\bar{B}_2=5.0$, $\bar{B}_3=6.0$, $\bar{B}_4=7.0$ (ppm)

The former case which will be called "Case a" is characterized by the lower standards for the water quality control, whereas the latter which will be named "Case b", the higher standards.

3.5.3 Calculation Cases

In prior to the calculations on the model, the following cases were preplanned.

(1) According to the difference in the predetermined available fresh waters, cases I, II, III, and IV are established.

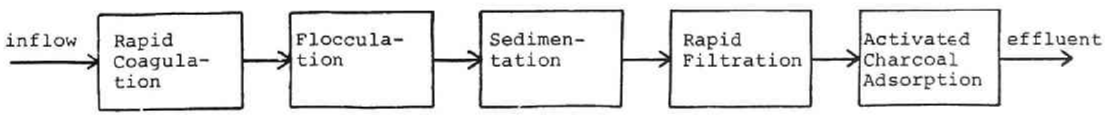


Fig. 3.5.5 Assumed Tertiary Treatment Processes

(ii) Case II was further subcategorized as Cases II-a and, II-b depending on whether either of the lower standards or the higher ones for the water quality control are employed. Let us take the results for Case II-a as our standard case and then make a comparative analysis of the results for the different cases.

3.5.4 Calculation Results

1) Mathematical Consideration of Efficiency of Search

Since our nonlinear model does not belong to a class of so called "concave programming", there is no guarantee of whether the obtained solution is a global minimum or a local one. The difficulty of this kind seems to be overcomed only by starting the search at a number of initial base points widely separated, and if the same value is found for all test cases, regarding this value as global and otherwise selecting the best as that value which can be termed global with some degree of confidence.

In this connection our attention is offered to the comparison between the results obtained from that search starting from the base point that has been found by solving the linearized-constraint model and those derived from that search initiating from an arbitrarily-chosen point. The study of Figure 3.5.6 which gives a good picture of the efficiencies of a number of searches attempted for the standard case (Case II-a) shows:

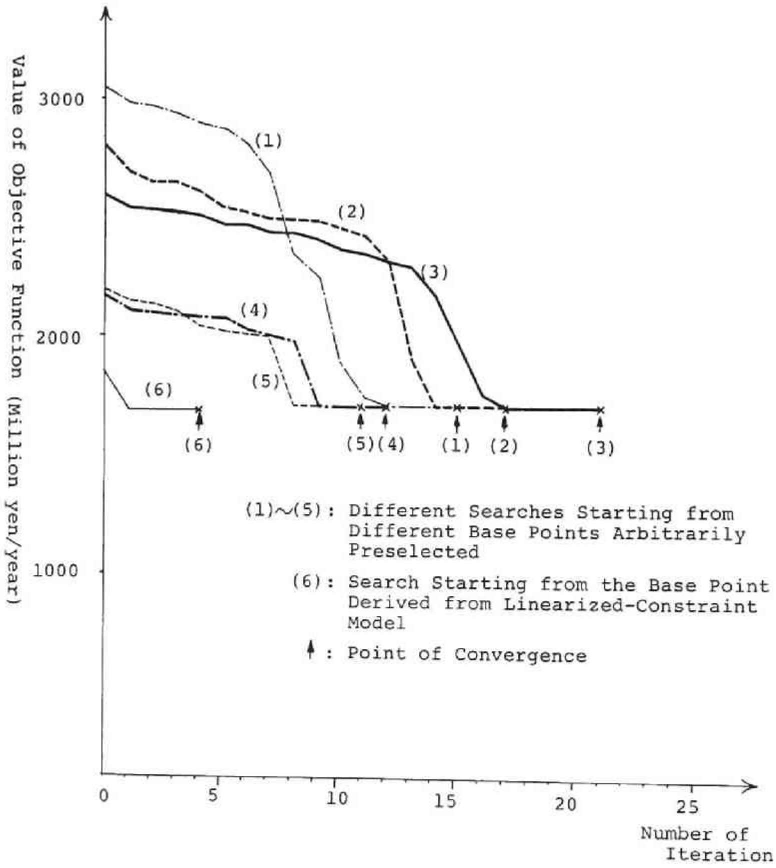


Fig. 3.5.6 Compared Results of Different Solution Searches

(i) A number of those searches which initiate from a different initial base points selected arbitrarily from the feasible solution set are seen to converge to two different local optima, one being obtained within 15 to 17 iterations and the other within 12 iterations. Those flat lines seen in the "tails" of the curves as illustrated in Figure 3.5.6 mean that the searches have already reached the local optima and the remainder of the searches follows the substantial ones and terminates only when the convergence is guaranteed to satisfy the predetermined tolerence.

This implies that (1) the substantial part of the searches ends up at the point corresponding to the joint of the tail (the left end of the flat line) and that ② the excessive accuracy predetermined for the check of convergence does not necessarily lead to the enhancement of the solution but it sometimes involves unnecessary additional searches.

(ii) There is little difference in value between the two optima, one being smaller by less than 0.2 percent than the other. This means that the difference can be considered negligible from the practical viewpoint.

(iii) That search starting from the base point that has been found beforehand by solving the linearized-constraint model is found to be the most efficient procedure, judged from the points of view that ① it takes a reduced number of iterations, that is, as small as 9 iterations (three-fourths to half of those for the aforementioned cases) including the dummy processes, or 2 iterations (one-fourth to one-eighth of those cases) excluding the processes, and that ② it leads to the minimum of the local optima which can be regarded as the global optimum or some good approximation to it. (The minimum thus obtained proved to be smaller by 2 percent than the second best local optimum.)

(iv) The above findings are shown to apply to Case I, where relatively large amounts of fresh water are assumed to be developed as compared to the standard case. Things turn out to be somewhat different for Case II-b, where the water quality control is assumed to be more strictly made than the standard case or for Cases III and IV where smaller amount of fresh water is assumed to be developed. That is to say that the initial base point derived from the linearized-constraint model proved to be identified exactly with the optima obtained from a number of iterative procedures which start from an arbitrarily-chosen point of the nonlinear-constraint model.

(v) It follows from this that the linearized-constraint model provides an efficient initial base point and that if the quality regulations are assumed to be strictly imposed on the stream, or if available quantity of fresh water is taken to be rigidly limited, it may be conceived as a good substitute for the nonlinear-constraint model.

2) Standard Case (Case II-a)

The results for the standard case are diagrammatically shown in Figure 3.5.7, from which the following may readily be understood.

(i) The reclamation system is implemented only in that downstream zone where some 35 percent of the wastewaters undergo the tertiary treatment, of which 100 percent are reused for industrial uses. This appears to be derived from the fact that relatively large amounts of water demanded for industrial uses in the downstream zone —— more than double as much as that demanded by the industries in the middle zone, guarantee remarkable decrease in the marginal costs associated with the increased implementation of the reclamation system as compared to the increased additional costs associated with that of the industrial water supply system.

Therefore one might easily understand that intensive implementation of the system proved to be more economical than otherwise. But at first blush the intensiveness of this kind does not seem to be reasonable for the purpose of alleviating water quality, because relatively large amount of water collected in both the upperstream and midstream would result in the deteriorated water quality in the downstream. But closer study shows that a large-scale implementation of the reclamation system to the extent that the renovated waters are utilized for both providing water and improving the quality of the waste-water effluents would lead to both the decrease in the amount of streamflow collection and the improvement in the quality of the downstream at the point of wastewater discharges. This means that the intensive implementation well serves for regulating water quality to reach the standard quality as well as minimizing the associated costs at the same time.

(ii) The quality of streamflow at the check-point of the upperstream or mid-stream zone is better than the standard quality. One exception to this is the downstream zone where the quality is found to be equal to the set standard. This suggests that on the upperstream and midstream such quality of streamflow better than the set standards is both attainable and efficient, judged from the point of view of minimizing the implementation costs. In this connection the reader is invited to the point that the linearization of the quality con-straints is justified as a good alternative to our model only if the quality of each check-point is found to be close to the given standards.

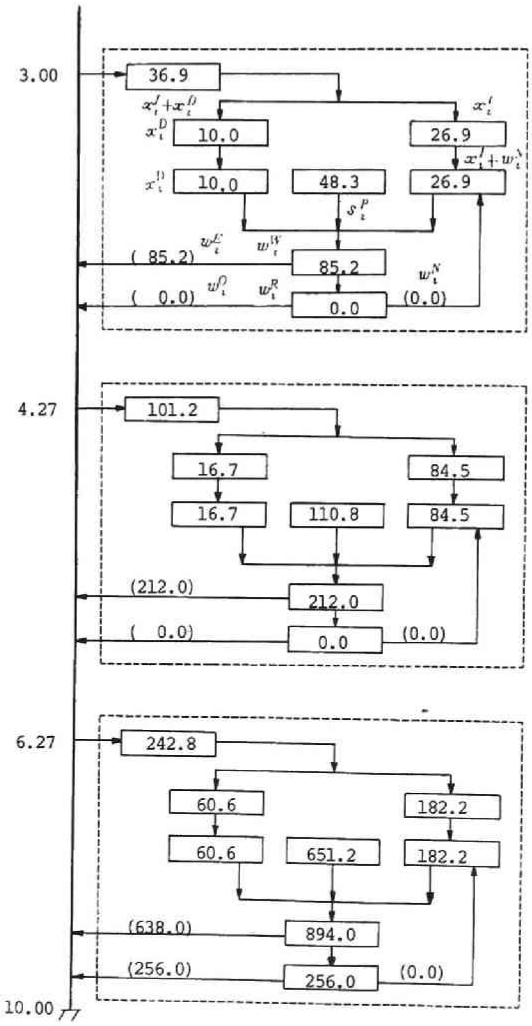


Fig. 3.5.7 Results for Standard Case II-a

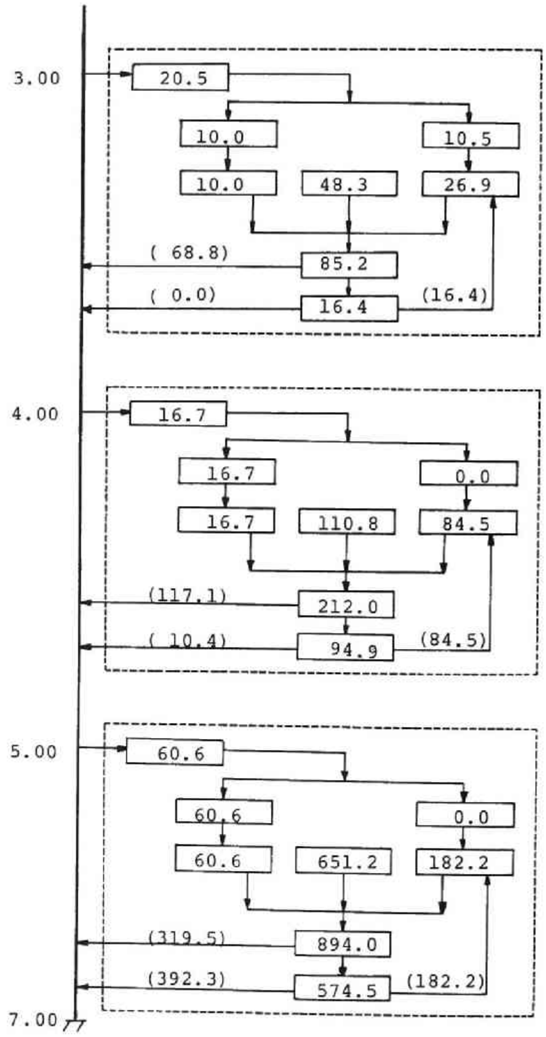


Fig. 3.5.8 Results for Case II-b

3) Comparative Study of Different Cases

(i) Figure 3.5.8 gives the results for Case II-b where the quality standards are taken to be severer than the standard case (Case II-a). From this it is evident that ① water utilization patterns are the same as those obtained for the standard case insofar as the upperstream and midstream are concerned; ② some amount of wastewaters are required to be renovated so as to improve the effluent quality; ③ the streamflow quality at each check-point is found to be equal to its set standard; ④ accordingly, this solution derived from our model is a posteriori shown to be identical to that obtained from the linearized-constraint model; ⑤ this is meant to imply that the latter can become a good substitute for the former in case high quality standards are a priori established.

(ii) Figure 3.5.9 shows the comparative analysis of Case I, Cases II-a (standard case), III and IV. From this we may easily understand that ① the needed amount of renovated waters decreases roughly in proportion to the increase in the amount of available fresh waters to be developed in the headwaters; ② the associated costs decrease with the increase in the available amount of fresh waters. (See Figure 3.5.10.) The cost curve exhibits a decreasing marginal cost associated with the implementation of the reclamation system. But if the cost for developing fresh water is included into the associated costs, the total costs tend to increase as larger amounts of fresh water are developed; ③ the streamflow quality at each check-point tends to become better as the quantity of available fresh water increases; ④ this means to imply that if relatively large amounts of fresh water are predetermined, it does not seem to be a good way to employ the linearized-constraint model instead of the original nonlinear-constraint model, whereas it serves as a good tool for finding some effective base point from which to start the solution search of our model.

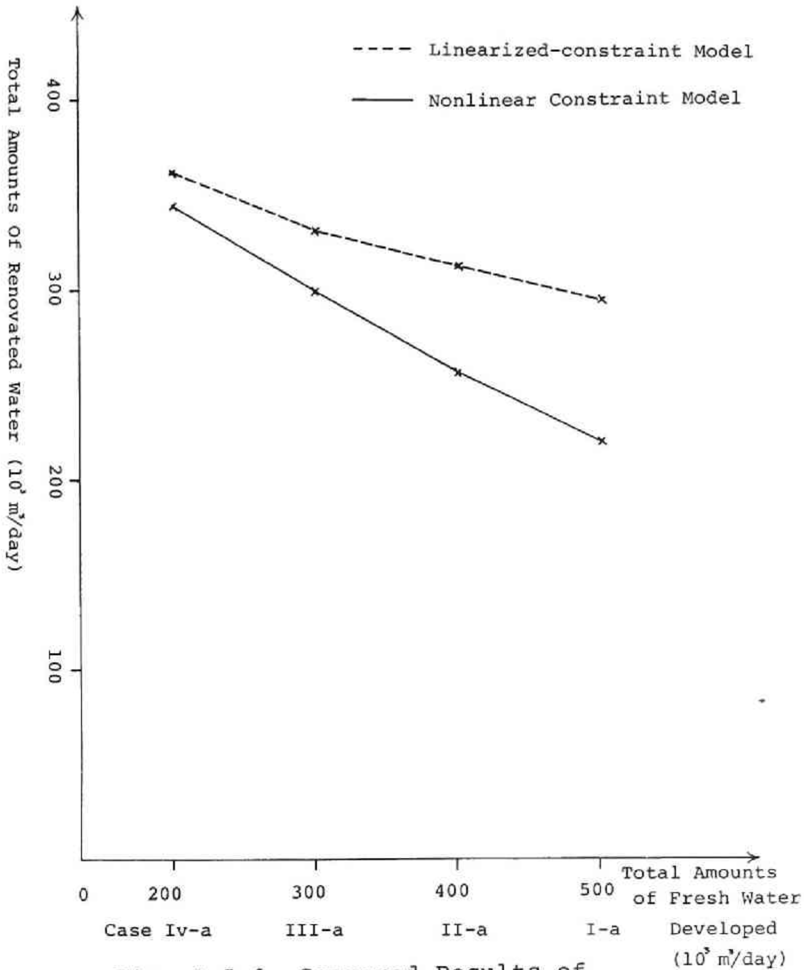


Fig. 3.5.9 Compared Results of Cases I-a, II-a, III-a and IV-a

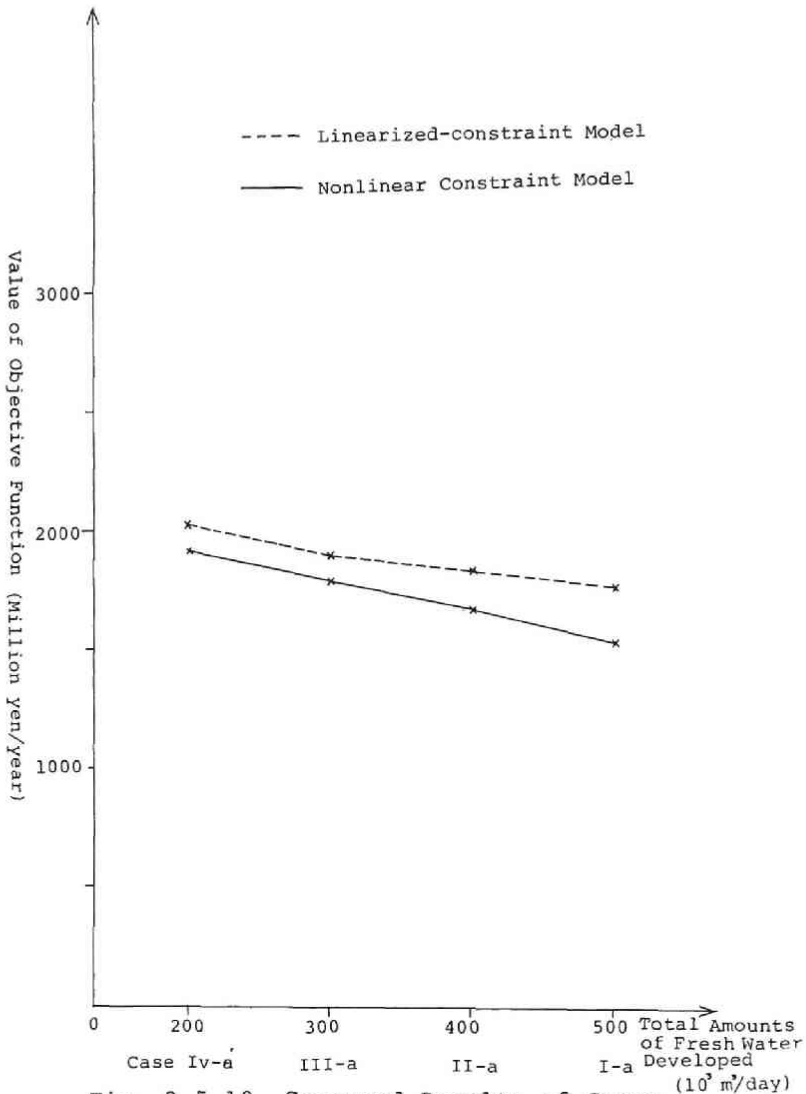


Fig. 3.5.10 Compared Results of Cases

3.6 Conclusion

The primary objectives of this study have been: ① to emphasize the importance of studying the development and utilization of an intra-basin water resources system; and ② to present a nonlinear programming model as an effective tool for analyzing the problem.

The major findings could be summarized as follows:

(i) The implementation of the reclamation system is found to be concentrated on the downstream zone where the entire demands for industiral uses are estimated exclusively higher than the other zones.

(ii) Accordingly intensive implementation of the reclamation system proved to serve for both regulating water quality to reach the set standards relative to the downstream and minimizing the associated costs, relative to the entire basin.

(iii) Any type of inter-zonal water distribution system should not be implemented.

(iv) The needed amount of renovated waters decreases roughly in proportion to the increase in the amount of available fresh waters to be developed in the headwaters.

(v) The associated costs decrease with the increase in the available amount of fresh waters. But if the cost for developing fresh water is included into the associated costs, the total costs tend to increase as larger amounts of fresh water are developed.

(vi) When the quality standards are taken to be higher (severer), a larger amounts of wastewaters are required to be renovated in the downstream zone, while the water utilization patterns taking place in the upperstream and midstream zones, remain the same as those obtained for the standard case.

(vii) The method of feasible directions combined with the penalty method developed by Fiacco and McCormic proved to be effective to the solution search of our model. If the quality regulation are assumed to be strictly imposed on the stream, or if available amount of fresh water is taken to be rigidly limited, the linearized-constraint model may be conceived as a good alternative to our model.

In light of these considerations, our conclusions, then, point to a need for more inclusive evaluations of intra-basin water resource development system. It is also clear that the presented nonlinear programming model will provide important information for planning and designing water resource systems.

The research needs uncoverd by this study stem first from the general lack of high-quality data on those costs associated with the implementation and running the concerned facilities, especially the reclamation system.

A second search need is to learn about the mechanism governing the stream-flow pollutions, which seems to be oversimplified in our model.

Greater knowledge in these areas can lead us closer to the goal of adequate assessment of the problem treated by this study.

References

1) Okada, N., and Kazuhiro, Y.: Nonlinear Programming Approach to the Water Assignment Problem for a Single River Basin, Proc. of JSCE, No.247, March 1976 (In Japanese).
2) Okada, N., and Kazuhiro, Y.: Nonlinear Programming Approach for the Analysis of Intra-basin, Multi-modal Water Utilization System, Memoirs of the Faculty of Engineering, Kyoto University (To Be Issued).
3) Ohuchi, T.: Systems Analysis of Single-basin, Multi-modal Water Utilization System, Graduation Thesis Presented to the Faculty of Engineering, Kyoto University, March, 1975 (In Japanese).
4) Fiacco, A.V., and McCormick, G.P.: The Sequential Unconstrained Minimization Technique for Non-linear Programming; A Primal-dual Method, Management Science, Vol.10, pp.360-366, 1964.
5) Fiacco, A.V., and McCormick, G.P.: Computational Algorithm for Sequential Unconstrained Minimization Technique for Non-linear Programming, Management Science, Vol.10, pp.601-617,1964.
6) Fiacco, A.V., and McCormick, G.P.: Extension of Sequential Unconstrainted and Extrapolation, Management Science, Vol.12, pp.816-818, 1966.
7) Zoutendijk, G.: Methods of Feasible Directions, Elsevier Publishing Company, 1960.
8) Rosen, J.B.: The Gradient Projection Method for Nonlinear Programming, Part I, Linear Constraints, J.Soc.Ind. Appl. Math., Vol.8, pp.181-217, 1960.
9) Rosen, J.B.: The Gradient Projection Method for Non-linear Programming, Part II, Nonlinear Constraints, J.Soc.Ind. Appl. Math., Vol.9, pp.514-532, 1961.
10) Kelley, J.E.: The Cutting Plane Method for Solving Convex Programs, J.Soc.Ind. Appl. Math., Vol.8, pp.703-712, 1960.
11) Wolfe, P.: Accelerating the Cutting-plane Method for Nonlinear Programming, J.Soc.Ind. Appl. Math., Vol.9, pp.481-488, 1961.

12) Glass, H., and Cooper, L.: Sequential Search; A Method for Solving Constrained Optimization Problems, J. Assoc. Computing Machinery, Vol.12(1), pp.71-82. 1965.
13) Hartley, H.O.: Non-linear Programming for Separable Objective Functions and Constraints, RAND Symposium on Mathematical Programming, RAND Corp. Rept. R-351, Santa Monica, Calif., 1960.
14) De Angelis, V.: On the Solution of a Structured Linear Programming Problem in Upper Bounded Variables; Optimization edited by R. Fletcher, Academic Press, 1969.
15) Ministry of Construction: Annual Bulletin of Stream Discharge, 1971-1974 (In Japanese).
16) Ministry of Construction: Annual Bulletin of Stream Quality, 1974 (In Japanese).
17) Hyogo Prefecture: Data on Discharge and Quality of Main Streams of Hyogo, Unpublished Material, 1974 (In Japanese).

Chapter 4 Decomposition-principle-based Analysis of Integrating Process of Inter-basin, Multi-modal Water Utilization System

4.1 Introduction

In the foregoing chapter the intra-basin, multi-modal water resources development system has been discussed in detail. The problem to which we now turn concerns the inter-basin, multi-modal water resources development system which is assumed to be a combined system of fresh-water development and reclamation systems. The major advantages of the fresh water development system may be summarized as follows.

(i) Although water resources are scarce in limited areas, especially in highly urbanized areas, it is also true that water would be available if the development is conducted in such a large scale that it includes other areas (basins) where water resources are still relatively abundant as compared to the low water demands generating there.

(ii) This means that the inter-basin development system that is characterized by the large-scale, inter-basin transfer of water from "surplus" to "deficit" areas, can be looked upon as a most promising and efficient measure for overcoming the water-shortage problem.

(iii) The low-flow augmentation by diverting streamflows from one basin to another contributes to the alleviation of the streamflow quality.

Chiefly from the viewpoints of (i) and (ii), we shall consider this system.

On the other hand recent technological progresses in the field of water resources have made it possible to introduce the reclamation systems. Major advantages of this system have already been discussed in the preceding chapter. Excluding other sources of supply such as desalted water, groundwater etc. and different ways of economizing of the demand side, the study is confined to the optimal combination of the above-cited two alternative systems, mainly because they are considered most promising and economical amongst different available measures. The term "optimal" is used to refer to the most economically combined system.

4.2 Integrating Planning Process Incorporated

At this point of discussion we need to observe that the inter-basin fresh-water development system is relatively concerned with the interests of area-wide regions, whereas the reclamation system to be implemented on a local basis, is exclusively concerned with the interests of a specified locality.[1)2)3)4)5)] This means that the problem of the optimal mix of the two different alternatives entails the efficient coordination of the conflicting interests between them.

In this connection it seems reasonable to assume the following.

(i) Any alternative of the fresh-water development system (whose exact definition will be given later) is selected by some planning agency of regional level whose jurisdiction covers some area-wide interests.

(ii) Any alternative of the reclamation system (whose exact definition will also be given later) is selected by some planning agency of local level whose responsibility mainly concerns the local interests.

(iii) Both kinds of alternatives are combined and readjusted by some central planning agency of national level whose responsibility lies in the coordination of different interests involved.

The above assumptions provide an important basis on which to construct our mathematical model. That is, the following three planning functions are to be incorporated into the model, i.e., (1) function that corresponds to the selection process of an appropriate alternative of the inter-basin fresh-water development system, (2) function that corresponds to the selection process of an appropriate alternative of the reclamation system and (3) function that corresponds to the integrating process where those alternatives derived from the above two processes are mutually combined to yield a global alternative. The term "appropriate" is used to mean that each function is required to select as economical an alternative as possible, assuming that the explicit criterion with which to check whether it is appropriate or not is limited to economical one. And the "global alternative" refers to that alternative synthesized from the two kinds of alter natives.

4.3 Model Formulation

4.3.1 Identification of the Problem

(i) The area-wide multiple river basins are treated where midstream and downstream areas in each basin are assumed to be grouped into one conceptual region which will be called hereafter "the demand region" of a given basin, and dams are assumed to be constructed at a number of sites in the upperstream valleys on each river.

(ii) The facilities to be explicitly considered are a set of dams to be constructed in each basin, inter-basin channels for streamflow diversions to be built between two adjacent basins. In each basin are also assumed collection facilities (1), filtration facilities (2) for industrial water supply and those for domestic water supply, wastewater treatment plant (1) and tertiary treatment plant (1). The numbers in the parentheses represent the numbers of facilities involved.

(iii) The objective function is taken to be the minimized total costs for constructing aforecited facilities.

4.3.2 Notation

1) Variables

u_{r_l} : amount of fresh water developed by dam l to be constructed in the headwaters of river r

x_r^I : amount of water treated in the filtration facilities for industrial supply to be constructed in the basin of river r

x_r^D : that for domestic water supply

$y_{rs}^T(y_{sr}^T)$: amount of river flow to be diverted from river r to s through the channel built between them. (This channel will be denoted by (rs) (or (sr)).

w_r^W : amount of wastewater treated by the wastewater treatment plant to be constructed in the basin of river r

w_r^R : amount of treated wastewater to be further renovated by the tertiary plant to be constructed in the basin of river r (=amount of renovated water to be reused for industrial uses)

2) Constants

C_{r_l} : upper bound on w_{r_l}

D^I_r : water demands for industrial uses in the demand region of river r

D^D_r : those for domestic uses

s^P_r : total amounts of existing water supply for industrial uses in the demand region of river r

α^I_r : unit cost associated with construction and maintenance of the industrial water filtration facilities in the demand region of river r

α^D_r : that of the domestic water filtration facilities

α^C_r : that of the collection facilities

$\beta^T_{rs}(\beta^T_{sr})$: that of the streamflow diversion channel (rs) (or(sr))

γ^W_r : that of the wastewater treatment plant

γ^R_r : that of the tertiary treatment plant

δ_{r_l} : that of dam l to be constructed on the upperstream of river r

L_r : number of dams to be constructed on river r

v : number of rivers

4.3.3 Constraints

In addition to the nonnegativity conditions of all the variables, we set the following conditions for each river basin $r=1,\cdots\cdots,v$.

In view of the limited capacity of each dam it follows

$$w_{r_l} \le C_{r_l} \quad (r=1,\cdots\cdots,v,\ l=1,\cdots\cdots,L_r) \qquad \text{(4.3.1)}$$

In each demand zone the following constraints hold with respect to the supplies of industrial, domestic and renovated waters as well as the wastewater treatment.

$$x^I_r + w^R_r \ge D^I_r \qquad \text{(4.3.2)}$$

$$x^D_r \ge D^D_r \qquad \text{(4.3.3)}$$

$$w^R_r \le w^W_r \qquad \text{(4.3.4)}$$

$$w^W_r \le D^I_r + D^D_r + s^P_r \qquad \text{(4.3.5)}$$

Futhermore for each river the following water-quantity balance should be satisfied.

$$\sum_l w_{r_l} + \sum_{s \in z_r} (y^T_{sr} - y^T_{rs}) - (x^I_r + x^D_r) \ge 0 \qquad \text{(4.3.6)}$$

where z_r denotes the set of those rivers adjacent to river r.

4.3.4 Objective Function

The objective function is taken to be the minimization of the total costs as expressed in the form:

$$\text{Minimize } Z=\sum_r \{\alpha^I_r x^I_r + \alpha^D_r x^D_r + \alpha^C_r (x^I_r + x^D_r)$$

$$+ \sum_{s \in z_r} (\beta^T_{sr} y^T_{sr} + \beta^T_{rs} y^T_{rs}) + \gamma^W_r w^W_r + \gamma^R_r w^R_r\} + \sum_{r=1}^{v} \sum_l \delta_{r_l} w_{r_l} \qquad \text{(4.3.7)}$$

4.3.5 Decomposition-principle-based Interpretation of Process of Integrating Two Planning Functions

The above-formulated model is a class of linear programming which can easily be handled by a well-known technique, i.e., the simplex method or its variants. But as explained before our another concern is to present a mathematical methodology for analyzing the optimal coordinating process of two planning functions involved in the above-specified water resources system. In accordance with this objective, we shall further dissect the above-formulated model by utilizing the principle of Decomposion due to Dantzig.

To begin, we define any set of solutions that satisfy Equation (4.3.1) as "an alternative of the fresh-water (development) system", and in likewise any set of solutions that satisfy Equations (4.3.2) through (4.3.5) as "an alternative of the reclamation system". In the following discussions where there is no danger of ambiguity, the terms "an alternative of the fresh water system" and "an alternative of the reclamation system" will be simply referred to as "a fresh-water alternative" and "a reclamation alternative", respectively.

Furthermore under the assumptions made in 4.2 the integrating processes of planning the combined system would be interpreted as follows.

Process 1

The fresh-water alternative and reclamation alternative are independently selected by the planning agencies of regional and local levels, respectively.

Process 2

Then the selected alternatives are sent to the central agency of national level to be optimally blended. That is to say that on receipt of these alternatives, the central agency is called upon to coordinate them, primarily because there is no guarantee that they will satisfy the water-quantity balance equation of (4.3.6), (Let this condition be named "feasibility condition".) and secondarily because thus combined system will not necessarily constitute the optimal alternative in the context of the area-wide economy as expressed by Equation (4.3.7). (Let this condition be named "optimal condition".) If some alternatives have already been stored at the central agency (these alternatives are called "old alternatives"), they are used to combine with the newly-selected ones (they are called "new alternatives"), thus producing a "new global alternative".

Process 3

If it proves to be inadequate from the viewpoints of either or both of the conditions, the central agency calls on the other agencies to propose another new alternative based on the information concerning how the preceding alternative is inadequate and how it should be modified.

Matters proceed as before, the global alternative being renewed and its value improved, until it is found to be most adequate (or optimal) in the context of the mentioned conditions.

We shall see later that the above assumptions give a well-illustrated interpretation of the decomposion-based solution technique applied to our model.[6)7)]

Since the set of the feasible solutions for Equation (4.3.1) is bounded convextly, any one of the feasible solutions can be represented by a convex combination of a certain number of the basic feasible solutions in this set. This is also true of any one of the feasible solutions that satisfy Equations (4.3.2) to (4.3.5). Let $\{w_{r_\ell}^{(p)}\}$ and $\{x_r^{(q)}\}$ represent any one of the basic feasible solutions to Equation (4.3.1) or to the set of Equations (4.3.2) to (4.3.5), respectively. Here $x_r^{(q)} = (x_r^{I(q)}, x_r^{D(q)}, w_r^{W(q)}, w_r^{R(q)})$. Then, any feasible solutions, $\{w_{r_\ell}\}$ and $\{x_r\}$, where $x_r = (x_r^I, x_r^D, w_r^W, w_r^R)$ can be expressed in terms of linear combinations of $\{w_{r_\ell}^{(p)}\}$ ($p=1$,

$\cdots\cdots, p_o$) and of $\{x_\tau^{(q)}\}$ $(q=1,\cdots\cdots,q_o)$, respectively. That is,

$$w_{\tau_l}=\sum_{p=1}^{p_o}\lambda^{(p)}w_{\tau_l}^{(p)} \qquad (4.3.8)$$

$$\sum_{p=1}^{p_o}\lambda^{(p)}=1 \qquad (4.3.9)$$

$$x_\tau=\sum_{q=1}^{q_o}\mu^{(q)}x_\tau^{(q)} \qquad (4.3.10)$$

$$\sum_{q=1}^{q_o}\mu^{(q)}=1 \qquad (4.3.11)$$

Substitution of Equations (4.3.8) and (4.3.10) into (4.3.6) yields

$$\sum_{p=1}^{p_o}\left(\sum_l u_{\tau_l}^{(p)}\right)\lambda^{(p)}-\sum_{q=1}^{q_o}\left(x_\tau^{I(q)}+x_\tau^{D(q)}\right)\mu^{(q)}+\sum_{s\in z_\tau}\left(y_{s\tau}^T-y_{\tau s}^T\right)\geqq 0 \qquad (4.3.12)$$

Likewise by substituting Equations (4.3.8) and (4.3.10) into (4.3.7), we obtain

$$\text{Minimize } z=\sum_{p=1}^{p_o}\left(\sum_{\tau=1}^{v}\sum_l\delta_{\tau_l}u_{\tau_l}^{(p)}\right)\lambda^{(p)}+\sum_{q=1}^{q_o}\sum_{\tau=1}^{v}\{(\alpha_\tau^I+\alpha_\tau^C)x_\tau^{I(q)}+(\alpha_\tau^D+\alpha_\tau^C)x_\tau^{D(q)}$$

$$+\gamma_\tau^W w_\tau^{W(q)}+\gamma_\tau^R w_\tau^{R(q)}\}\mu^{(q)}+\sum_{\tau=1}^{v}\sum_{s\in z_\tau}\left(\beta_{s\tau}^T y_{s\tau}^T+\beta_{\tau s}^T y_{\tau s}^T\right) \qquad (4.3.13)$$

If we set

$$z_1^{(p)}=\sum_{\tau=1}^{v}\sum_l\delta_{\tau_l}u_{\tau_l}^{(p)} \qquad (4.3.14)$$

$$z_2^{(q)}=\sum_{\tau=1}^{v}\{(\alpha_\tau^I+\alpha_\tau^C)x_\tau^{I(q)}+(\alpha_\tau^D+\alpha_\tau^C)x_\tau^{D(q)}+\gamma_\tau^W w_\tau^{W(q)}+\gamma_\tau^R w_\tau^{R(q)}\} \qquad (4.3.15)$$

$$\sigma_\tau^{(p)}=\sum_l u_{\tau_l}^{(p)} \qquad (4.3.16)$$

$$\xi_\tau^{(q)}=x_\tau^{I(q)}+x_\tau^{D(q)} \qquad (4.3.17)$$

then Equations (4.3.12) and (4.3.13) are rewritten as

$$\sum_{p=1}^{p_o}\sigma_\tau^{(p)}\lambda^{(p)}+\sum_{q=1}^{q_o}\xi_\tau^{(q)}\mu^{(q)}+\sum_{s\in z_\tau}\left(y_{s\tau}^T-y_{\tau s}^T\right)\geqq 0 \qquad (4.3.18)$$

$$\text{Minimize } z=\sum_{p=1}^{p_o}z_1^{(p)}\lambda^{(p)}+\sum_{q=1}^{q_o}z_2^{(q)}\mu^{(q)}+\sum_{\tau=1}^{v}\sum_{s\in z_\tau}\left(\beta_{s\tau}^T y_{s\tau}^T+\beta_{\tau s}^T y_{\tau s}^T\right) \qquad (4.3.19)$$

Observing that in Equations (4.3.18) and (4.3.19) $\sigma_\tau^{(p)}$, $\xi_\tau^{(q)}$, $z_1^{(p)}$, $z_2^{(q)}$, $\beta_{s\tau}^T$ and $\beta_{\tau s}^T$ are all constants while $\lambda^{(p)}$, $\mu^{(q)}$, $y_{s\tau}^T$, $y_{\tau s}^T$ all variables, and taking account of Equations (4.3.9) and (4.3.11) which $\lambda^{(p)}$ and $\mu^{(q)}$ respectively should satisfy, we know that Equations (4.3.18), (4.3.19), (4.3.9), (4.3.11) and the nonnegativity conditions of all the variables concerned constitute a linear programming problem. We shall call this problem "the Master Problem", which is restated as:

<u>Master Problem</u>

$$\text{Minimize } z=\sum_{p=1}^{p_o}z_1^{(p)}\lambda^{(p)}+\sum_{q=1}^{q_o}z_2^{(q)}\mu^{(q)}+\sum_{\tau=1}^{v}\sum_{s\in z_\tau}\left(\beta_{s\tau}^T y_{s\tau}^T+\beta_{\tau s}^T y_{\tau s}^T\right) \qquad (4.3.19)$$

subject to

$$\sum_{p=1}^{p_o}\sigma_\tau^{(p)}\lambda^{(p)}-\sum_{q=1}^{q_o}\xi_\tau^{(q)}\mu^{(q)}+\sum_{s\in z_\tau}\left(y_{s\tau}^T-y_{\tau s}^T\right)\geqq 0 \qquad (4.3.18)$$

$$\sum_{p=1}^{p_o} \lambda^{(p)} = 1 \quad \cdots\cdots (4.3.9)$$

$$\sum_{q=1}^{q_o} \mu^{(q)} = 1 \quad \cdots\cdots (4.3.11)$$

$$(\lambda^{(p)} , \mu^{(q)} , y^T_{sr} , y^T_{rs} \geqq 0)$$

This master problem can be considered a mathematical expression of the process 2, or what comes to the same thing, the problem of finding a set of proper weights to be assigned to both the new and old alternatives, thereby producing a new global alternative.

The new global alternative thus obtained is warranted to satisfy the feasibility condition. The optimality condition which is called "simplex criterion" can be checked as follows.

$$\bar{z}_1^{(p)} = z_1^{(p)} - \pi^{[k]} \sigma^{(p)} \quad \cdots\cdots (4.3.20)$$

$$\bar{z}_2^{(q)} = z_2^{(q)} - \pi^{[k]} \xi^{(q)} \quad \cdots\cdots (4.3.21)$$

where

$$\sigma^{(p)} = [\sigma_1^{(p)}, \cdots\cdots , \sigma_v^{(p)}, 1, 0]^t$$

$$\xi^{(q)} = [\xi_1^{(q)}, \cdots\cdots , \xi_v^{(q)}, 0, 1]^t$$

$$\pi^{[k]} = [\pi_1^{[k]}, \cdots\cdots , \pi_m^{[k]}, \pi_{m+1}^{[k]}, \pi_{m+2}^{[k]}] \ .$$

In the above $\pi^{[k]}$ represents a set of simplex multipliers corresponding to the optimal basis for the master problem of the k-th iteration. That is, $\pi^{[k]}$ is calculated as

$$\pi^{[k]} = \omega^{[k]} (B^{[k]})^{-1} \quad \cdots\cdots (4.3.22)$$

where $(B^{[k]})^{-1}$ represents the inverse matrix of the optimal basis for the master problem of the k-th iteration, $\omega^{[k]}$ its corresponding coefficient parameters $z_1^{(p)}$ and/or $z_2^{(p)}$ in the objective function (4.3.19).

The values of the simplex multiplers derived from the master problem of the k-th iteration can be considered the information provided by the central agency for the remaining two agencies as to the way to cut the total costs. This mechanism corresponds to the process 3.

On receipt of this information the two agencies other than the central one are called upon to modify their own alternatives. We shall further explain this mechanism in terms of the decomposion principle.

$$\bar{z}_1^{(p)} = \sum_{r=1}^{v} \sum_{l} \bar{\delta}_{r_l} w_{r_l}^{(p)} - \pi_{m+1}^{[k]} \quad \cdots\cdots (4.3.23)$$

$$\bar{z}_2^{(q)} = \sum_{r=1}^{v} \{ (\bar{\alpha}_r^I + \alpha_r^C) x_r^{I(q)} + (\bar{\alpha}_r^D + \alpha_r^C) x_r^{D(q)} + \gamma_r^W w_r^{W(q)} + \gamma_r^R w_r^{R(q)} \} - \pi_{m+2}^{[k]} \quad \cdots\cdots (4.3.24)$$

$$\bar{\delta}_{r_l} = \delta_{r_l} - \pi_r^{[k]} \quad \cdots\cdots (4.3.25)$$

$$\bar{\alpha}_r^I = \alpha_r^I - \pi_r^{[k]} \quad \cdots\cdots (4.3.26)$$

$$\bar{\alpha}_r^D = \alpha_r^D - \pi_r^{[k]} \quad \cdots\cdots (4.3.27)$$

The theory of the simplex method shows that if the minimal values of $\bar{z}_1^{(p)}$ and $\bar{z}_2^{(q)}$ are found to be nonnegative, then the global condition is satisfied and this new global alternative is shown to be identical to our desired solution.

It also tells that otherwise that new alternative, whether $u_{r_l}^{(p)}$ ($r=1,\cdots\cdots,v$; $l=1,\cdots\cdots,L_r$; $p=p_o+1$) or $x_r^{(q)}$ ($r=1,\cdots\cdots,v$; $q=q_o+1$), which yields the most negative value among them should be added to those old alternatives which constituted the old master problems. Here we obtain the problems of finding the minima of both $\bar{z}_1^{(p)}$ and $\bar{z}_2^{(q)}$ subject to Equations (4.3.1), and Equations (4.3.2-1) through (4.3.5-1), respectively which can be stated as follows.

Subproblem I

$$\text{Minimize } \bar{z}_1^{(p)} = \sum_{r=1}^{v} \sum_{l} \bar{\delta}_{r_l} u_{r_l}^{(p)} - \pi_{m+1}^{(k)} \qquad (4.3.28)$$

$$u_{r_l}^{(p)} \leq C_{r_l} \quad (u_{r_l}^{(p)} \geq 0 , p_o = p_o + 1) \qquad (4.3.1)$$

Subproblem II

$$\text{Minimize } \bar{z}_2^{(q)} = \sum_{r=1}^{v} \{ (\bar{\alpha}_r^I + \bar{\alpha}_r^C) x_r^{I(q)} + (\bar{\alpha}_r^D + \bar{\alpha}_r^C) x_r^{D(q)} + \gamma_r^W w_r^{W(q)} + \gamma_r^R w_r^{R(q)} \} - \pi_{m+2}^{(k)} \qquad (4.3.29)$$

$$x_r^{I(q)} + w_r^{R(q)} \geq D_r^I \qquad (4.3.2\text{-}1)$$

$$x_r^{D(q)} \geq D_r^D \qquad (4.3.3\text{-}1)$$

$$w_r^{R(q)} \leq w_r^{W(q)} \qquad (4.3.4\text{-}1)$$

$$w_r^{W(q)} \leq D_r^I + D_r^D + s_r^P \qquad (4.3.5\text{-}1)$$

$$(x_r^{I(q)} , x_r^{D(q)}, w_r^{R(q)}, w_r^{W(q)} \geq 0 , q = q_o + 1)$$

The above two problems can be considered the mathematical representations of the process 1.

Finally the optimality condition is checked with the following criterion:

$$H = \min (\bar{z}_1^{*(p)} , \bar{z}_2^{*(q)}) \qquad (4.3.30)$$

and $\quad H \geq 0 \qquad (4.3.31)$

where $\bar{z}_1^{*(p)}$ and $\bar{z}_2^{*(q)}$ represent the optimal values of the objective functions of the above subproblems.

If the above condition is found to be invalid, the new alternatives corresponding to $\bar{z}_1^{*(p)}$ and $\bar{z}_2^{*(q)}$ are sent to the central agency and matters proceed as before until the above condition holds. These mechanisms can be interpreted as the mathematical expressions of the process 3.

We have now shown that the decomposition-based solution algorithm as explained in the above can be conceived as a modelled integrating process of the two different kinds of alternatives.

4.4 Case Study on Southern Part of Hyogo Prefecture

4.4.1 Regional Setting

The conceptualized system of inter-basin, multi-modal water resources development is diagrammed in Figure 4.4.1, which shows that the inter-basin system consists of 5 intra-basin systems, i.e., the Chigusa, Ibogawa, Yumesaki, Ichikawa and Kakogawa Rivers.

4.4.2 Model Data

1) Estimated Water Demands

On the basis of the projected increments of population and growth of pro-

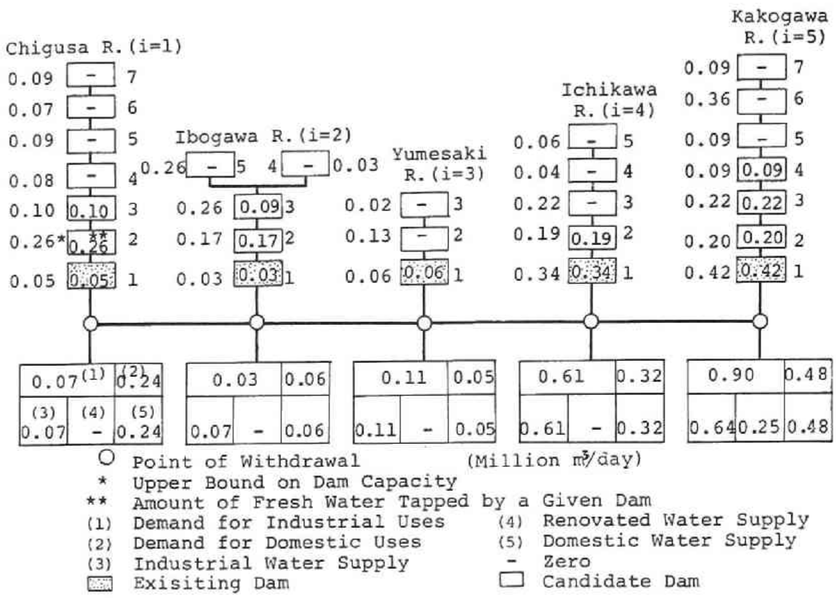


Fig. 4.4.1 Model Diagram and Calculation Results of Standard Case

duction over the period ranging from 1965 through 1985, predicted per-capita demands and per-unit-production demands at the time of 1985, the predicted water demands were estimated as tabulated in Table 4.4.1.[8)9)]

(Million m³/day)

Name of River Basin	Industrial Demand	Domestic Demand
Kakogawa	0.90	0.48
Ichikawa	0.61	0.32
Yumesaki	0.11	0.05
Ibogawa	0.03	0.06
Chigusa	0.07	0.24

Table 4.4.1 Projected Water Demands

2) Estimated Cost Curves

The same data as used in the preceding chapter were employed. (For further details see Figure 3.4.2 to 3.4.4.) The nonlinearities involved in some of the cost curves were overcomed as follows.

3) Modified Linear Programming Approach

Whether the model is solved straightfowardly as a linear programming problem or approached by use of the decomposition principle, the solution algorithm developed is required to be based on the simplex method which is only applicable to a class of linear programming problems. To get around this difficulty we shall develop the modified linear programming approach for this kind of problem.

Brief descriptions of this procedure will follow.

$$\text{Minimize } z = \sum_{j=1}^{n_1} e_j x_j + \sum_{j=n_1+1}^{n_2} f_j(x_j) \quad \cdots\cdots (4.4.1)$$

$$\sum_{j=1}^{n_2} a_{ij} x_j \geq b_i \qquad (x_j \geq 0,\ i=1, \cdots\cdots, m) \quad \cdots\cdots (4.4.2)$$

where $f_j(x_j)$ are continuous nonlinear functions of a single variable x_j.

We first find some large values M_j such that

$$0 \leq x_j \leq M_j \quad \cdots\cdots (4.4.3)$$

Assume that we select r_j+1 points, x_{j_k}, where $x_{j_0}=0$, $x_{j_1} < x_{j_2} < \cdots\cdots < x_{j_r} = M_j$ in the interval $0 \leq x_j \leq M_j$. Now for each k we compute $f_{j_k} = f_{j_k}(x_{j_k})$. Imagine that for each k we next connect the points (x_{j_k}, f_{j_k}) and $(x_{j_{k+1}}, f_{j_{k+1}})$ by a straight line. We thus obtain the dashed curve shown in Figure 4.4.2, which is then an approximation to $f_j(x_j)$ in the interval $0 \leq x_j \leq M_j$. The dashed curve or polygonal approximation to $f_j(x_j)$ will be denoted by $\hat{f}_j(x_j)$. When x_j lies in the interval $x_{j_k} \leq x_j \leq x_{j_{k+1}}$ we are approximating $f_j(x_j)$ by $\hat{f}_j$, where

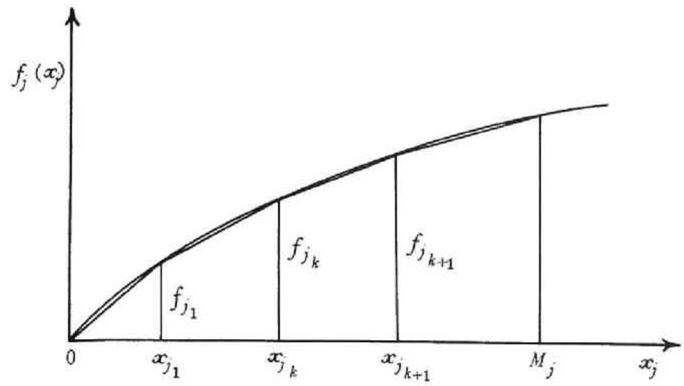


Fig. 4.4.2 Piecewise Linear Approximation

$$\hat{f}_j = f_j + \frac{f_{j_{k+1}} - f_{j_k}}{x_{j_{k+1}} - x_{j_k}} (x_j - x_{j_k})$$

$$= e_{j_k} x_j + d_{j_k} \qquad \cdots\cdots (4.4.4)$$

$$e_{j_k} = \frac{f_{j_{k+1}} - f_{j_k}}{x_{j_{k+1}} - x_{j_k}} \qquad \cdots\cdots (4.4.5)$$

$$d_{j_k} = f_j - e_{j_k} x_{j_k} \ . \qquad \cdots\cdots (4.4.6)$$

Procedure 1

Initially let x_j $(j = n_1 + 1, \cdots, n_2)$ be assumed to lie in the given intervals $x_{j_{k_j}} \leq x_j \leq x_{j_{k_j+1}}$ (where k_j are properly pre-selected for each x_j.)

Procedure 2

Solve the following linear programming.

$$\text{Minimize } z = \sum_{j=1}^{n_1} e_j x_j + \sum_{j=n_1+1}^{n_2} e_{j_k} x_j \qquad \cdots\cdots (4.4.7)$$

$$\sum_{j=1}^{n_2} a_{ij} x_j \geq b_j \quad (x_j \geq 0, \ i = 1, \cdots\cdots, m) \qquad \cdots\cdots (4.4.2)$$

Procedure 3

Check whether the solution to the above problem lies in the preassumed intervals for x_j. Such being the case, the procedures terminate and the above solution proves to be the desired solution. If not, we proceed to the next procedure.

Procedure 4

Update the intervals x_j's lie and repeat the same procedures from 2 to 4 until the aforementioned condition holds.

4.4.3 Calculation Results

1) Linear-Programming-Based Analysis

Before resorting to the application of the decomposition principle to the model, we shall analyze the results obtained from the model by utilizing the simplex method in a straight-forward way. Some of the conspicuous points are summarized. (See Figure 4.4.1.)

(i) Fresh waters tapped in the headwaters of the Chigusa, Ibogawa are conveyed to both the Yumesaki and Ichikawa Rivers. Another inter-basin streamflow diversion system is implemented between the Ichikawa and Kakogawa Rivers in order to transport stream-flows of the Kakogawa to the Ichikawa River.

(ii) Large-scale construction of the reclamation system is exclusively concentrated on the basin of the Kakogawa River where 70 percent of its water demands for industrial uses are covered by the system and the remaining 30 percent by the fresh water development system.

(iii) One of the major reasons for this kind of concentrated implementation of the reclamation system is that its implementation cost exhibits a mold of nonlinearity with respect to its scale, or what comes to the same, the scales of economy. This implies that the implementation of the reclamation system, where necessary, would be concentrated exclusively on one or some of those regions of high water demands.

(iv) The parametric programming analysis in which the associated costs are parameterized to see the effect of its change on the outputs, shows that overestimated costs for the fresh water development system, for instance, tend to lead to a higher coverage of the reclamation system and vice verse. (See Figure 4.4.3.)

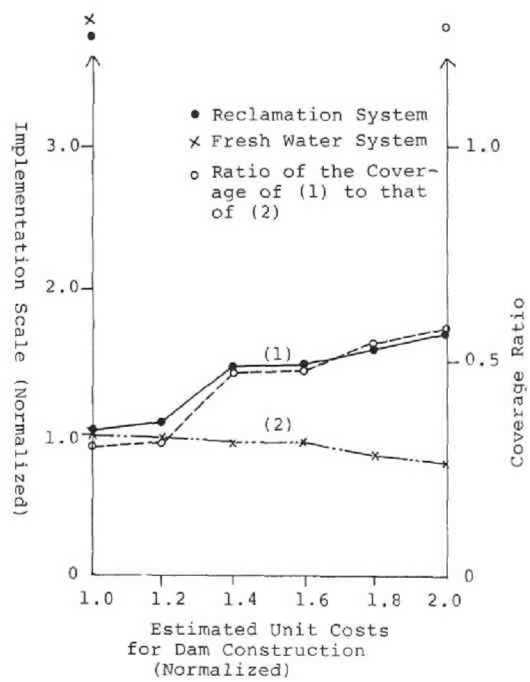


Fig. 4.4.3 Calculated Implementation Scale for Different Unit Costs Predetermined

(v) In case the water demands are parameterized, the scale of the reclamation system expands roughly in proportion to the increment in the water demands, whereas the scale of the fresh water development system remains almost the same. (See Figure 4.4.4.) From this we may reason that the overestimated water demands tend to place the reclamation system in a more advantageous position, because of increased advantage of the scales of economy related to the system.

2) Decomposition Principle-Based Analysis

Keeping the above findings in mind we proceed to the analysis of the intergrating planning processes involved in the inter-basin, dual-modal water resources development system.

(a) Changing Pattern of Alternative Selected at Each Stage

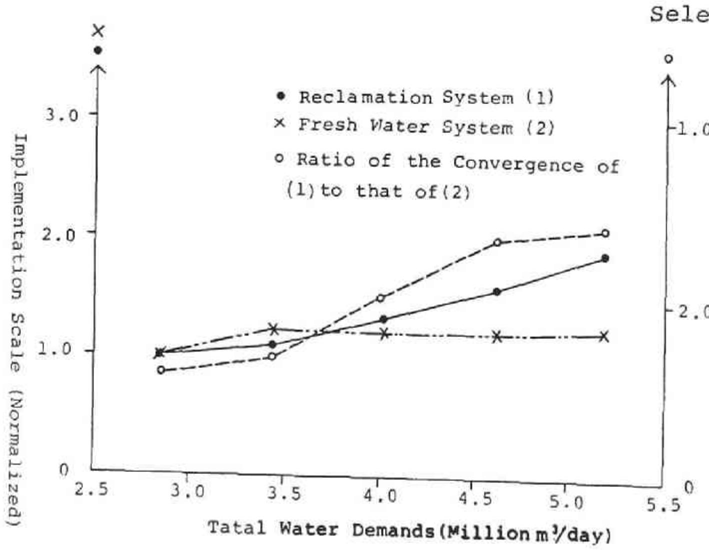


Fig. 4.4.4 Calculated Implementation Scale for Different Water Demands Predetermined

The study of Figure 4.4.5 reveals the following:

(i) A crucial difference seen in the changing pattern of alternatives selected at each stage lies between the former stages (from stages 1 to 4) and the latter ones (from stages 5 to 7). That is to say that in the former stages those scales of both the fresh water and reclamation systems represented by the selected new alternatives exhibit comparatively drastic changes for each stage as compared to those changes in the latter stages. This seems to imply that the

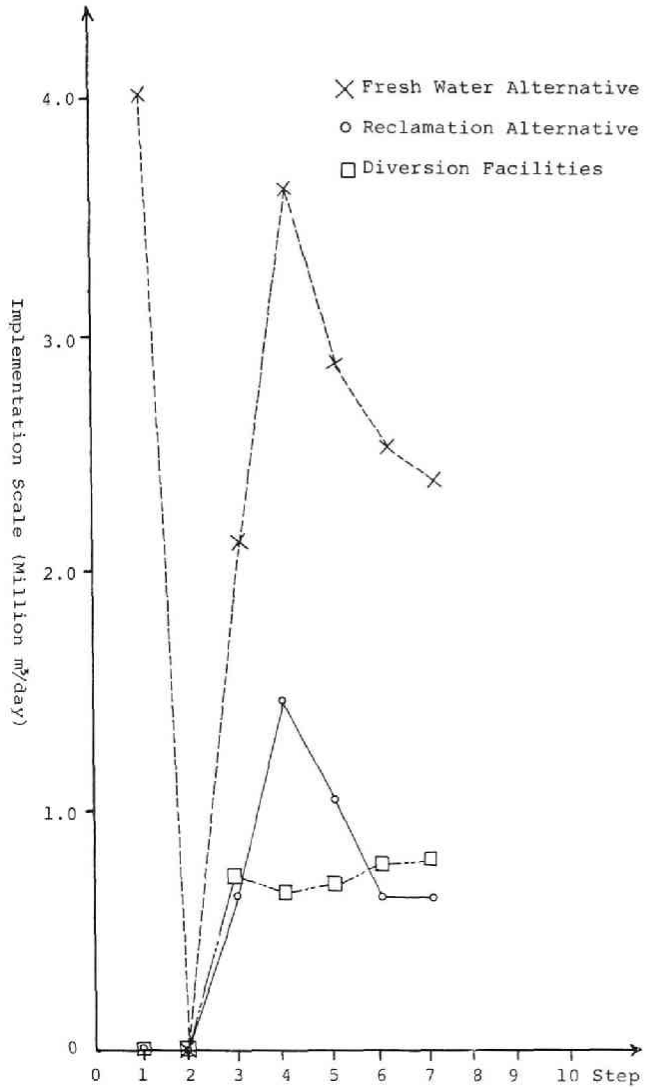


Fig. 4.4.5 Changing Pattern of Sub-alternatives Selected at Each Stage

former stages can be interpreted as the process of roughing out the percentages of the coverages of the total demands by either of the two kinds of systems, whereas the latter stages correspond to the refining process where detailed assignments of the afore-examined coverages by either of the systems to each basin are developed step by step.

(ii) In this connection it should be observed that the quantity of the water demands covered by the reclamation system amounts to 0.64 million m^3/day and that covered by the fresh water system 2.22 million m^3/day. Notably the marginal cost for that scale of the reclamation system from as much as 0.50 million m^3/day to more than that amount becomes as low as the sum of both unit implementation costs of the fresh water system (whose scales total 2.22 million m^3/day) and those of the diversion channels involved. This means that the basic coverages by two kinds of systems can be approximated by selecting those dam sites whose implementation costs are estimated at 15 yen/m^3 or less than that and by identifying those basins where the water demands are estimated to be more than 0.50 million m^3/day.

(b) Changing Patterns of New Alternatives Selected at Each Stage and Weights Assigned to Them

Comparative analysis of Table 4.4.2 shows:

(i) The assigned weights to the new alternatives added to the master program for each stage are considered to represent their relative importances in the set of alternatives involved.

(ii) The weight assigned to that reclamation alternative selected at stage 3 has proven to be equal to 1.0 at the final stage, which implies that this alternative system can be considered the reclamation subsystem itself which constitues the optimal water resources system.

(iii) Those fresh water alternatives selected at stages 3 and 7 are combined

Kind of Alternatives	No. of (p), (q)		Initially Introduced at step :	Step 1	2	3	4	5	6	7
Fresh Water Alternatives	(p)	1	1	1.0	0.75	0.08		-	-	-
		2	1	-	-	-		-	-	-
		3	2	*	0.25	-	-	-	-	-
		4	3	*	*	0.92	1.0	0.80	0.72	0.61
		5	4	*	*	*	-	-	-	-
		6	5	*	*	*	*	0.20	-	-
		7	6	*	*	*	*	*	0.28	-
		8	7	*	*	*	*	*	*	0.39
Reclamation Alternatives	(q)	1	1	1.0	1.0	-	-		-	-
		2	1	-	-		-	-	-	-
		3	1	-	-	-		-	-	-
		4	1	-	-	-	-	-	-	-
		5	1	-	-	-	-	-		-
		6	2	*	-	-	-	-	-	-
		7	3	*	*	1.0	0.77	1.0	1.0	1.0
		8	4	*	*	*	0.23	-	-	-
		9	5	*	*	*	*	-	-	-
		10	6	*	*	*	*	*	-	-
		11	7	*	*	*	*	*	*	-

- denotes zero

* that alternative not yet introduced into the Master Problem

Table 4.4.2 Calculated Weights for Different Steps

into the fresh-water subsystem which constituted the optimal water resources system. By observing that the weight assigned to that alternative selected at stage 3 is 0.61 whereas that selected at stage 7 is 0.39, we might easily reason that the basic characteristics of the optimal (global) system is determined by that alternative selected at stage 3 and that some modifications of the structure are made by that alternative selected at stage 7.

(iv) This leads us to the conclusion that those alternatives selected at stage 3, whether fresh-water or reclamation alternative, represent the basic structure of the optimal system. In other words stage 3 proves to be the crucial point in the roughing-out phase (from stages 1 to 4).

(c) Changing Pattern of the Value of the Objective Function

The above findings can be validated also by the study of the changing pattern of the value of the objective function as illustrated in Figure 4.4.6. That is, drastic decreases are seen from stages 1 to 3 as compared to the slight decreases seen from stages 4 to 7. This implies that the phase of the framework-making of the optimal system ranges from stages 1 to 3 or so and that the phase that follows can be thought of as the refining processes.

4.5 Conclusion

The study included in this chapter concerns the interbasin, multi-modal water development system which consists of two kinds of subsystems, i.e., fresh-water development system and reclamation system. Thereby our attention was devoted to the analysis of the integrating process of the two different planning functions involved in combining fresh-water and reclamation alternatives presented respectively by their concerned agencies. Then thus specified problem was modelled by linear programming and further analyzed by use of the decomposition principle. To illustrate the potential of the model a case study on

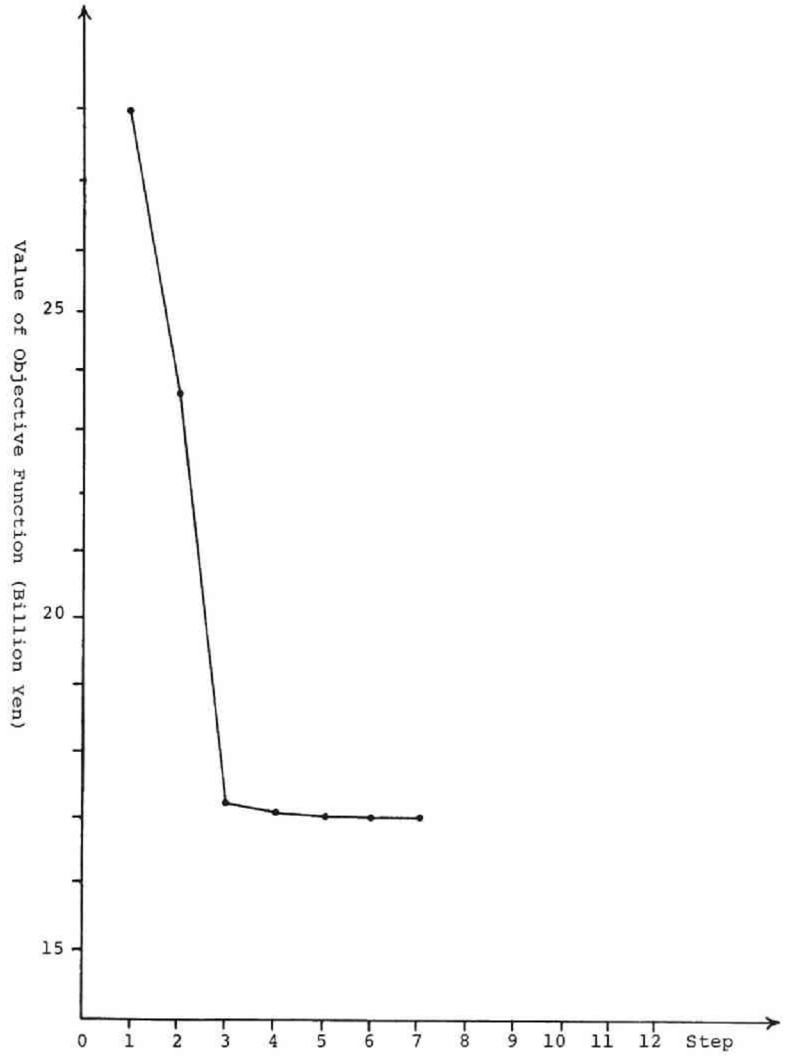


Fig. 4.4.6 Changing Pattern of Objective Function at Each Stage

the southern part of Hyogo Prefecture was made, from which the following points have been made clear.

(i) The reclamation system should be implemented exclusively in those regions where relatively high water demands are estimated, mainly owing to the scales of economy involved in the implementation of this system. This kind of region is found to be the basin of the Kakogawa River in our case study.

(ii) The inter-basin steramflow diversion systems should be implemented to convey streamflows from both the Chigusa and Ibogawa Rivers to both the Yumesaki and Ichikawa Rivers, and also from the Kakogawa to Ichikawa Rivers.

(iii) The parametric programming analyses in which either of the associated costs or the estimated water demands are parametrized, reveals that the overestimated implementation costs and/or the overestimated water demands tend to place the reclamation system in a more advantageous position.

(iv) The decomposition-based analysis reveals that the earlier stages of the integrating process correspond to the phase of roughing out both the total amounts of the water supply to be covered by the fresh-water development system and those shared by the reclamation system. In view of the fact that this kind of coordination was found to contribute largely to the enhancement in the integration, we might well conclude that if such roughing-out mechanism is given primary consideration in prior to the detailed examination of the optimal global system, it would reduce a vast amount of work needed to follow it.

The above findings seem to provide important information for the planner who examines the possibilities of the inter-basin, multi-modal water resources development system as one of the practical strategies to overcome the water-shortage crisis in the years immediately to follow.

Lastly the study is incomplete in that:
① The quality of streamflows involved should be included into the objectives of the study.
② Even within a single basin there seems to be much difference in localism, hydrological conditions, economical and social situations, etc. Accordingly further study should be continued where each of the concerned river basins are divided into several subregions.

In light of these considerations another feature of the inter-basin, multi-modal water development system will be considered in next chapter.

References

1) Okada, N., and Haruna, M.: Study on a Mathematical Model for Water Assignment Planning in the Large-scale Water Resource Development, Proc. of JSCE, No.211, March, 1973 (In Japanese).
2) Okada, N., and Haruna, M.: A Systems Analysis for the Planning of Facilities in Inter-basin Water Distribution Problems, the Memoirs of the Faculty of Engineering, Kyoto University, Vol.XXXIV, Part 3, PP.235-261, July, 1974.
3) Okada, N., and Yoshikawa, K.: A System Analysis for Water Distribution Problem —— Combined System of Extensive Development of Water Resources and Water Reuse, Proc. of JSCE, No.247, March, 1976 (In Japanese).
4) Okada, N.: Study on Area-wide Water Utilization System, Master Thesis Presented to the Faculty of Engineering, Kyoto University, March, 1972 (In Japanese).
5) Yoshinaga, K.: Systems Analysis of Inter-basin, Dual-modal Water Utilizations, Master Thesis Presented to the Faculty of Engineering, Kyoto University, March, 1974 (In Japanese).
6) Dantzig, G.B.: Linear Programming and Extensions, Princeton Univ. Press, 1963.
7) Dantzig, G.B., and Wolfe, P.: The Decomposition Algorithm for Linear Programs, Econometrica, Vol.29, pp.767-778, 1961.
8) Prefecture of Hyogo: Water Resources Planning Perspectives of Hyogo Prefecture, 1973 (In Japanese).
9) Ministry of Construction: Annual Bulletin of Stream Discharge, 1971-1974 (In Japanese).
10) Ministry of Construction: Annual Bulletin of Stream Quality, 1974 (In Japanese).
11) Prefecture of Hyogo: Data on Discharge and Quality of Main Streams of Hyogo, Unpublished Material, 1974 (In Japanese).

Chapter 5 Nonlinear Goal Programming Approach for Coordinated Attainment of Multi-goal, Inter-basin, Multi-modal Water Utilization System

5.1 Introduction

Practitioners and researchers of the art of water management have increasingly come to the realization that allocation and control of this resource in the high population areas where the demand for water is growing rapidly, requires considerably greater skills than have been utilized in the past if effective management is to be attained. Furthermore with the increasing awareness of the importance of environmental quality, planning and construction agencies are asked to broaden environmental considerations. As such, the evolution of area-wide, multi-modal water resources planning to include multiple means of water-usage modes on an integrated regional basis has become a matter of increasing concern in the area of resource management. As a result the central question now in the planning of water resources development and utilization is how to blend the attainment of the required goals related to developing sources of water as well as to controlling water quality. In light of these considerations the primary concern of this paper is the proper coordination of the attainments of multiple goals involved in the planning of this kind.[1)2)3)4)5)] For this purpose a mathematical methodology by use of goal programming will be highlighted as a means to approach the problem.

5.2 Multiple Goals In the Planning of Water Utilization System

The system to which our discussion is addressed is the water resource system of inter-basin development and dual modal utilization. (This system will be called "inter-basin system" for short.) Among several goals involved in the planning of this system the following two seem to be given first consideration.

(i) to secure a sufficiency of fresh water provisions for the water users on each river basin by means of developing dams on a cross-basin basis and diverting river flow from one stream to another,

(ii) to attain an efficiency of economy on a cross-basin, area-wide basis.

In addition to the above goals, chiefly owing to the increased awareness of the need to make concerted attacks on the problems of the environment, the planner in the field of water resources development is called upon to include some forms of water quality considerations into the principal goals for the planning. From this point of view, let us deal with the reclamation system as a means to improve water quality of the receiving waters. In this regard, it seems better to divide one river basin into multiple zones by allowing for different localisms of hydrology and water uses for the reasons that follow. ①Some forms of recycling usages of water resources have been and are being in operation on each stream where stream waters which are collected and used on the upperstream are partially returned back with waste loads to the receiving stream and they are re-collected and reused and returned back to the receiving water body on the midstream and/or downstream. ② In this viewpoint it is keenly needed to establish an efficient water recycling system combined with the reclamation system.

To sum up in dealing with the above-analyzed system, the following goals should be given major consideration.

(i) to secure a sufficiency of the provisions of water, whether fresh or renovated, as much as possible for the use in each zone,

(ii) to attain an efficiency of economy as much as possible on a cross-basin, area-wide basis,

(iii) to improve the water quality of each stream as much as possible.

Herewith attention needs to be paid to the fact that these three different goals can be attained to full extent only at the expense of the others. Stated otherwise, the more economical alternative is seeked for, the less amounts of renovated waters seem to be needed, and the more deteriorated quality of stream water should be permitted. Moreover the more sufficient provisions are called upon in a given zone, the less amounts of water are available in the other zones, whether they may be located on the same stream or not. Accordingly the planner would be rather concerned with such an alternative that assures coordinated attainments of the multiple goals than he would seek for that alternative which maximizes the attainment of a single goal. We shall dissect this type of problem in this paper.

5. 3 Goal Programming

5. 3. 1 Preliminary Discussion

Goal Programming is a special extension of linear programming. Though the interest for it is rather new, the literature is already copious.[6)7)8)9)]

Recently T.Fushimi and T.Yamaguchi demonstrated that the introduction of the L-type utility function into the formulation of the model as a mathematical representation of the trade-off relations between the concerned goals, leads to the problem of finding such a solution which can be thought of as well-balanced attainments of those goals.[10)] We decided to make use of this approach developed by T.Fushimi and T.Yamaguchi, primarily because our main concern is to find that alternative which assures well-balanced attainments of our goals.

In this regard it seems to be of vital importance that their approach bases its frame of modelling on linear programming and in this context it is not applicable to those cases where the goals to be set takes a mold of nonlinearity, as is exactly the case with our model. (This point will explicitly be described later.)

In light of these considerations our secondary concern is placed on exploring Fushimi-Yamaguchi's approach and developing a new type of goal programming which is applicable to the problem involving nonlinear goal constraints. As will be seen, the authors present a modified goal programming approach based on the cutting plane method developed by Kelley[11)] and will demonstrate its applicability to the non-linear-type problems.

5. 3. 2 Permitted-level and Satisfied-level of aGoal

To begin, let us introduce the notions of the "permitted-level" and the "satisfied-level" with reference to a given goal. The term "permitted level" is defined as the level such that the planner is determined to accept any alternative that assures the attainments of the concerned goals to the extent equal to, or larger than it, but otherwise he would never accept that alternative. The term the "satisfied level" is used to mean that after taking account of various conditions, such as the case where one specified goal would be attempted to reach its full attainment by confining the remaining goals to be achieved at their permitted-levels, the planner becomes willing to accept that level of alternative as a satisfactory one, if not optimal.

5.3.3 Specification of the Method

For brevity and clarity of explanation this will be deferred to the formulation of our model.

5.4 Model Formulation

5.4.1 Identification of the Problem

(i) The area-wide, multiple river basins are considered where we assume that by taking account of local differences in water usages, hydrology, legislative and economic boundaries, etc., the area of each river basin is a priori divided into a couple of subareas which we call "downstream (demand)zone", "midstream zone", "upperstream zone" and the like.

(ii) In the valleys up the rivers and above the upstream zones are constructed dams to develop fresh water.

(iii) Each zone on a given river is considered a geographical unit for water utilization in which a dual-modal water utilization system — a combined system for utilizing both fresh water and renovated water — is assumed to be implemented.

(iv) There are two types of water users in each zone, i.e., industrial water and domestic water users.

(v) As shown in Figure 5.4.1, in each zone water is collected from the nearest stream running through the area and then undergoes purification at two different utilization plants (one for industrial water supply and the other for domestic water supply), and is provided for both kinds of water users. The used water with waste loads are carried by the sewers to a wastewater treatment plant and then undergo primary and secondary treatments. The effluents from the plant are partially returned back through an outlet to the same stream (receiving water body) at a point further down from the point of withdrawal, the remainder of which being treated by the tertiary treatment plant and then returned back to the receiving water body at the point near the wastewater-discharge point, and partially provided again for the industrial water use.

(vi) Between a given water body and its adjacent ones is constructed a diversion channel through which river flows are diverted from one stream to another, if necessary.

(vii) It is also admitted that between any two of all the zones, whether located on the same stream or different streams, can be constructed a distribution aqueduct through which some portions of purified industrial water are conveyed from one zone to the other.

(viii) At a given point on the stream in each zone which assumed to be located most downstream in each zone water quality of the stream is required to meet the set standard. We stand on the premise that water quality is checked with a single parameter,i.e.,BOD ppm.

(ix) The construction costs are rendered to the amortization cost per annum.

(x) The set goals are the following three: ① minimized total costs associated with the construction and operation of the concerned facilities (cost-goal); ② maximized attainment of water supply to meet the projected water demands of each zone (supply-goal); ③ maximized attainment of water quality conservation at the check-point on the stream in each zone (quality-goal).

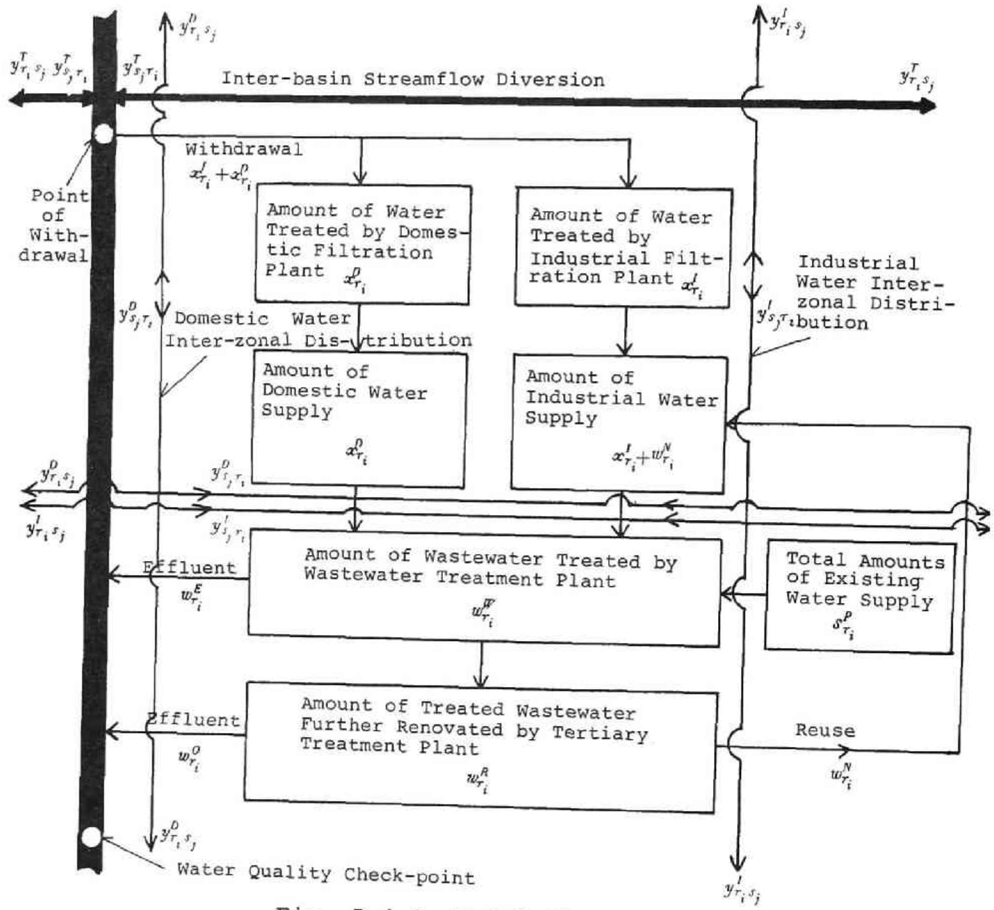


Fig. 5.4.1 Model Diagram

5.4.2 Notation

1) Variables

w_{r_l} : capacity of dam l to be constructed on the upperstream valley of river r

$x^I_{r_i}$: amount of water treated by the filtration facility for industrial water supply to be constructed in zone i on river r

$x^D_{r_i}$: that for domestic water supply

$S^I_{r_i}$: amount of water supply for industrial use

$S^D_{r_i}$: that for domestic uses

$y^I_{r_i s_j}$: amount of purified industrial water to be distributed from zone i on river r to zone j on river s

$y^D_{r_i s_j}$: that of purified domestic water

$y^T_{r_i s_j}$: amount of river flow to be diverted from river r in zone i to river s in zone j through the channel built between them (This channel will be denoted by channel$(r_i s_j)$).

$w^W_{r_i}$: amount of wastewater treated by the wastewater treatment plant to be constructed in zone i on river r

$w^E_{r_i}$: amount of treated wastewater to be discharged into the receiving water body r

$w^R_{r_i}$: amount of treated wastewater to be further renovated by the tertiary plant

$w^N_{r_i}$: amount of renovated water to be reused by industry

$w^O_{r_i}$: amount of renovated water to be discharged into the receiving water body

$Q^L_{r_i}$: average annual discharge at the downstream-most point on river r in zone i

$Q^U_{r_i}$: that at the upperstream-most point

$B^L_{r_i}$: average BOD at the downstream-most point (check-point) on river r in zone i

$B^U_{r_i}$: that at the upperstream-most point

$B^T_{r_i s_j}$: average BOD of streamflow diverted by channel $(r_i s_j)$

ε^C, η^C: deviational variables related to cost-goal

$\varepsilon^I_{r_i}(\varepsilon^D_{r_i})$: those related to industrial-(domestic-)supply-goal for river r

$\eta^I_{r_i}(\eta^D_{r_i})$: the same above

$\varepsilon^B_{r_i}, \eta^B_{r_i}$: those related to quality-goal for river r

2) Constants

c_{r_l}: upper bound on the capacity of dam l on river r

$q^1_{r_i}$: minimum discharge requirement of that part of river r running through zone i to be reserved to maintain the natural functions of the stream

$q^2_{r_i}$: amount of streamflow of river r already being collected by existing water users in zone i

$q^3_{r_i}$: inflow from tributary streams running through zone i into river r

$s^P_{r_i}$: total amounts of existing water supply for industrial and domestic uses in zone i on river r

$b^3_{r_i}$: average BOD of $q^3_{r_i}$

b^E: average BOD of the effluent from the wastewater treatment plant (same value for each zone)

b^O: average BOD of renovated waters (same value for each zone)

$\overline{B}_{r_i}$: BOD standard prescribed a priori at the check-point downstream-most of river r in zone i

G^C, g^C : satisfied and permitted levels of cost-goal, $(\lambda^C = g^C - G^C)$

$G^I_{r_i}, g^I_{r_i}$: those of industrial-supply-goal for zone i on river r $(\lambda^I_{r_i} = G^I_{r_i} - g^I_{r_i})$

$G^D_{r_i}, g^D_{r_i}$: those of domestic-supply-goal $(\lambda^D_{r_i} = G^D_{r_i} - g^D_{r_i})$

$G^B_{r_i}, g^B_{r_i}$: those of quality-goal $(\lambda^B_{r_i} = g^B_{r_i} - G^B_{r_i})$

$\alpha^C_{r_i}$: unit cost associated with construction and maintenance of the collection facilities in zone i on river r

$\alpha^I_{r_i}$: that of the industrial water filtration facilities in zone i on river r

$\alpha^D_{r_i}$: that of the domestic water filtration facilities in zone i on river r

$\beta^I_{r_i s_j}(\beta^I_{s_j r_i})$: that of the industrial water distribution aqueduct which conveys water from zone i (j) to j (i)

$\beta^D_{r_i s_j}(\beta^D_{s_j r_i})$: that of the domestic water distribution aqueduct

$\beta^T_{r_i s_j}(\beta^T_{s_j r_i})$: that of the streamflow diversion chanel $(r_i s_j)$ (or $(s_j r_i)$)

$\gamma^W_{r_i}$: that of the wastewater treatment plant

$\gamma^E_{r_i}$: that of the wastewater effluent facilies

$\gamma^R_{r_i}$: that of the tertiary treatment plant

$\gamma^O_{r_i}$: that of the renovated-water effluent facilities

$r^N_{r_i}$: that of the renovated-water distribution conduits

δ_{r_l} : that of dam l to be constructd on the upperstream of river r

L_r : number of dams to be constructed on river r

m_r, m_s: number of zones on river $r(s)$

v : number of rivers

5.4.3 Physical and Technical Constraints

In addition to the nonnegativity conditions of all the variables the following physical and technical constraints are incorporated into the model. For brevity of explanation, otherwise stated, the following constraints hold for $r=1, \cdots\cdots, v$; $l=1, \cdots\cdots, L_r$; $i=1, \cdots\cdots, m_r$; $j=1, \cdots\cdots, m_s$.

Each dam to be constructed is limited its capacity as

$$w_{r_l} \leqq c_{r_l} \qquad (5.4.1)$$

Available amount of streamflow to be collected is constrained as

$$x^I_{r_i} + x^D_{r_i} \leqq Q^U_{r_i} - q^1_{r_i} - q^2_{r_i} + \sum_{s \in z_{r_i}} (y^T_{s_j r_i} - y^T_{r_i s_j}) \qquad (5.4.2)$$

where z_{r_i} denotes the set of those rivers running through the zones adjacent to zone i on river r .

For water supply for industrial uses it follows

$$s^I_{r_i} \leqq x^I_{r_i} + \sum_{s \in z_{r_i}} y^I_{s_j r_i} - \sum_{s \in z_{r_i}} y^I_{r_i s_j} + w^N_{r_i} \qquad (5.4.3)$$

Likewise for water supply for domestic uses,

$$s^D_{r_i} \leqq x^D_{r_i} + \sum_{s \in z_{r_i}} y^D_{s_j r_i} - \sum_{s \in z_{r_i}} y^D_{r_i s_j} \qquad (5.4.4)$$

The amount of wastewater treated by the wastewater treatment plant in zone i is so limited

$$w^W_{r_i} \leqq s^I_{r_i} + s^D_{r_i} + s^P_{r_i} \qquad (5.4.5)$$

And the amount of renovated water treated by the tertiary plant in zone i and that of renovated water to be reused in this zone are so constrained

$$w^R_{r_i} = w^W_{r_i} - w^E_{r_i} \qquad (5.4.6)$$

$$w^N_{r_i} = w^R_{r_i} - w^O_{r_i} \qquad (5.4.7)$$

The average annual discharge of river r at the withdrawal point most upperstream in zone i is governed by the streamflow condition of the nearest upward zone as follows.

$$Q^U_{r_i} = \sum_{i' \in U_{r_i}} (q^3_{r_{i'}} + w^E_{r_{i'}} + w^O_{r_{i'}} - x^I_{r_{i'}} - x^D_{r_{i'}}) + \sum_{l=1}^{L_r} w_{r_l} + \sum_{\substack{j' \in U_{s_{j'}}, \\ i' \in U_{r_i},\ s \in z_{r_i}}} (y^T_{s_{j'} r_{i'}} - y^T_{r_{i'} s_{j'}}) \qquad (5.4.8)$$

where $U_{r_i}(U_{s_j})$ denotes the set of those zones located further upward from zone $i(j)$.

On substituting Equation (5.4.8) into (5.4.2), we get

$$x^I_{r_i} + x^D_{r_i} + \sum_{i' \in U_{r_i}} (x^I_{r_{i'}} + x^D_{r_{i'}}) \leqq \sum_{i' \in U_{r_i}} q^3_{r_{i'}} - (q^1_{r_i} + q^2_{r_i})$$

$$+ \sum_{s \in z_{r_i}} (y^T_{s_j r_i} - y^T_{r_i s_j}) + \sum_{\substack{j' \in U_{s_j}, \\ i' \in U_{r_i},\ s \in z_{r_i}}} (y^T_{s_{j'} r_{i'}} - y^T_{r_{i'} s_{j'}}) + \sum_{i' \in U_{r_i}} (w^E_{r_{i'}} + w^O_{r_{i'}}) + \sum_{l=1}^{L_r} w_{r_l} \qquad (5.4.9)$$

For preparation for the formulation of the quality-goals, let us assume a case where the water quality (BOD) of river r at the check-point downstream-most in zone i is merely confined to meet the set standard and not explicitly controlled as the quality-goal. It is expressed as

$$B^L_{r_i} \leqq \overline{B}_{r_i} \quad (5.4.10)$$

where

$$B^L_{r_i} = \frac{1}{Q^L_{r_i}} \{ B^U_{r_i} (Q^U_{r_i} - x^I_{r_i} - x^D_{r_i} - \sum_{s \in z_{r_i}} y^T_{r_i s_j}) + b^3_{r_i} q^3_{r_i} + b^E w^E_{r_i} + b^O w^O_{r_i} + B^T_{s_j r_i} y^T_{s_j r_i} \} . \quad (5.4.11)$$

Let us also assume here

$$Q^U_{r_i} = Q^L_{r_{i*}} \quad (5.4.12)$$

$$B^U_{r_i} = B^L_{r_{i*}} \quad (5.4.13)$$

$$B^T_{s_j r_i} = B^L_{s_{j*}} \quad (5.4.14)$$

where $i^*(j^*)$ denotes that zone located one-zone above zone $i(j)$. (See Figure 5.4.2.)

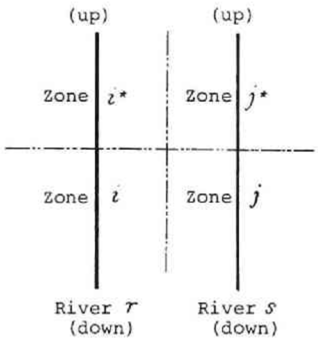


Fig. 5.4.2 Illustrated Relation Between Zone i on river r and Zone j on river s

By substituting Equations (5.4.12), (5.4.13) and (5.4.14) into (5.4.10) and (5.4.11), we obtain the following recurrence relation with respect to i:

$$B^L_{r_i} = \frac{1}{Q^L_{r_i}} \{ B^L_{r_{i*}} (Q^L_{r_{i*}} - x^I_{r_i} - x^D_{r_i} - \sum_{s \in z_{r_i}} y^I_{r_i s_j}) + b^3_{r_i} q^3_{r_i}$$

$$+ b^E w^E_{r_i} + b^O w^O_{r_i} + B^L_{s_{j*}} y^T_{s_j r_i} \} \leqq \overline{B}_{r_i} \quad (5.4.15)$$

Since both $B^L_{r_{i*}}$ and $Q^L_{r_{i*}}$ are also required to meet Equations (5.4.11) and (5.4.15), respectively, we know that Equation (5.4.15) is a nonlinear inequality.

5.4.4 Goal Constraints

The goal concerned with the total construction and operation costs are fomulated as

$$e = \sum_{r=1}^{v} \sum_{i} \{ \alpha^I_{r_i} x^I_{r_i} + \alpha^D_{r_i} x^D_{r_i} + \alpha^C_{r_i} (x^I_{r_i} + x^D_{r_i}) + \sum_{s \in z_{r_i}} (\beta^I_{r_i s_j} y^I_{r_i s_j} + \beta^I_{s_j r_i} y^I_{s_j r_i} + \beta^D_{r_i s_j} y^D_{r_i s_j} + \beta^D_{s_j r_i} y^D_{s_j r_i}$$

$$+ \beta^T_{r_i s_j} y^T_{r_i s_j} + \beta^T_{s_j r_i} y^T_{s_j r_i}) + \gamma^W_{r_i} w^W_{r_i} + \gamma^R_{r_i} w^R_{r_i} + \gamma^E_{r_i} w^E_{r_i} + \gamma^O_{r_i} w^O_{r_i} + \gamma^N_{r_i} w^N_{r_i} \} + \sum_{r=1}^{v} \sum_{l} \delta_{r_l} u_{r_l} \quad (5.4.16)$$

$$e - \varepsilon^C + \eta^C = G^C \quad (5.4.17)$$

$$e \leqq g^C \quad (5.4.18)$$

For the goals concerned with the amount of supply for industrial uses in zone i, it follows

$$S^I_{r_i} + \varepsilon^I_{r_i} - \eta^I_{r_i} = G^I_{r_i} \quad (5.4.19)$$

$$S^I_{r_i} \geqq g^I_{r_i} \quad (5.4.20)$$

$$\frac{\varepsilon^C}{\chi^C} = \frac{\varepsilon^I_{r_i}}{\chi^I_{r_i}} \quad (5.4.21)$$

Likewise the goals concened with the amount of supply for domestic uses in zone i, it follows

$$S^D_{r_i} + \varepsilon^D_{r_i} - \eta^D_{r_i} = G^D_{r_i} \quad (5.4.22)$$

$$S^D_{r_i} \geqq g^D_{r_i} \quad (5.4.23)$$

$$\frac{\varepsilon^C}{\lambda^C}=\frac{\varepsilon^D_{r_i}}{\lambda^D_{r_i}} \quad \cdots\cdots \quad (5.4.24)$$

Now we proceed to consider that case where the control of water quality at a given check-point in zone i, is incorporated into the model as a goal. By equating the BOD standard $\overline{B}_{r_i}$ with the permitted-level of the quality-goal, and on setting higher quality values as its satisfied-level, the goal is formulated as follows.

$$\frac{1}{Q^L_{r_i}}\{B^L_{r_i *}(Q^L_{r_i *}-x^I_{r_i}-x^D_{r_i}-\sum_{s \in z_{r_i}} y^T_{r_i s_j})+b^3_{r_i} q^3_{r_i} + b^E w^E_{r_i} + b^O w^O_{r_i} + B^L_{s_j *} y^T_{s_j r_i}\} - \varepsilon^B_{r_i} + \eta^B_{r_i} = G^B_{r_i} \quad (5.4.25)$$

$$\frac{1}{Q^L_{r_i}}\{B^L_{r_i *}(Q^L_{r_i *}-x^I_{r_i}-x^D_{r_i}-\sum_{s \in z_{r_i}} y^T_{r_i s_j})+b^3_{r_i} q^3_{r_i} + b^E w^E_{r_i} + b^O w^O_{r_i} + B^L_{s_j *} y^T_{s_j r_i}\} \leqq q^B_{r_i} \quad \cdots\cdots \quad (5.4.26)$$

$$\frac{\varepsilon^C}{\lambda^C}=\frac{\varepsilon^B_{r_i}}{\lambda^B_{r_i}} \quad \cdots\cdots \quad (5.4.27)$$

In view of the fact that $Q^L_{r_i}$ is constrained by variables such as $x^I_{r_i}$, $x^D_{r_i}$ etc., it is clear that the goal constraints (5.4.25) and (5.4.26) are nonlinear restrictions.

The problem that faces us now is the mold of nonlinearity involved in the goal constraints, because our ordinary goal programming approach is only applicable to that case where the goal constraints and physical and technical constraints as well as the objective function are all expressed in linear forms.

So as to tackle with this difficulty in a straightfoward way, a modified (nonlinear) goal programming approach will be presented later by the authors. Before that we shall proceed to the formulation of the objective function to complete our model.

5.4.5 Objective Function

Minimize $Z=\varepsilon^C$ ······ (5.4.28)

where ε^C can be replaced by any one of $\varepsilon^I_{r_i}$, $\varepsilon^D_{r_i}$ or $\varepsilon^B_{r_i}$ ($i=1,\cdots\cdots,m_r$; $r=1,\cdots\cdots,v$).

5.4.6 Formulated Model

<u>Objective function</u>

Minimize $Z=\varepsilon^C$ ······ (5.4.28)

<u>Technical and Physical Constraints</u>

For $l=1,\cdots\cdots,L_r$; $r=1,\cdots\cdots,v$; $i=1,\cdots\cdots,m_r$; $j=1,\cdots\cdots,m_s$, the following constraints hold:

$$w_{r_l} \leqq C_{r_l} \quad \cdots\cdots \quad (5.4.1)$$

$$x^I_{r_i}+x^D_{r_i} \leqq Q^U_{r_i} - q^1_{r_i} - q^2_{r_i} + \sum_{s \in z_{r_i}} (y^T_{s_j r_i} - y^T_{r_i s_j}) \quad \cdots\cdots \quad (5.4.2)$$

$$S^I_{r_i} \leqq x^I_{r_i} + \sum_{s \in z_{r_i}} y^I_{s_j r_i} - \sum_{s \in z_{r_i}} y^I_{r_i s_j} + w^N_{r_i} \quad \cdots\cdots \quad (5.4.3)$$

$$S^D_{r_i} \leqq x^D_{r_i} + \sum_{s \in z_{r_i}} y^D_{s_j r_i} - \sum_{s \in z_{r_i}} y^D_{r_i s_j} \quad \cdots\cdots \quad (5.4.4)$$

$$w^W_{r_i} \leqq S^I_{r_i} + S^D_{r_i} + S^P_{r_i} \quad \cdots\cdots \quad (5.4.5)$$

$$w^R_{r_i} = w^W_{r_i} - w^E_{r_i} \quad \cdots\cdots \quad (5.4.6)$$

$$w^N_{r_i} = w^R_{r_i} - w^O_{r_i} \quad \cdots\cdots \quad (5.4.7)$$

$$Q^U_{r_i} = \sum_{i' \in U_{r_i}} (q^3_{r_{i'}} + w^E_{r_{i'}} + w^O_{r_{i'}} - x^I_{r_{i'}} - x^D_{r_{i'}}) + \sum_{l=1}^{L_r} w_{r_l} + \sum_{\substack{j' \in U_{s_{j'}},\\ i' \in U_{r_i},\ s \in Z_{r_i}}} (y^T_{s_{j'} r_{i'}} - y^T_{r_{i'} s_{j'}}) \quad \cdots\cdots (5.4.8)$$

<u>Linear Goal Constraints</u>

$$e - \varepsilon^C + \eta^C = G^C \quad \cdots\cdots (5.4.17)$$

$$e \leqq g^C \quad \cdots\cdots (5.4.18)$$

$$e = \sum_{r=1}^{v} \sum_i \{\alpha^I_{r_i} x^I_{r_i} + \alpha^D_{r_i} x^D_{r_i} + \alpha^C_{r_i}(x^I_{r_i} + x^D_{r_i}) + \sum_{s \in z_{r_i}} (\beta^I_{r_i s_j} y^I_{r_i s_j} + \beta^I_{s_j r_i} y^I_{s_j r_i} + \beta^D_{r_i s_j} y^D_{r_i s_j} + \beta^D_{s_j r_i} y^D_{s_j r_i}$$
$$+ \beta^T_{r_i s_j} y^T_{r_i s_j} + \beta^T_{s_j r_i} y^T_{s_j r_i}) + \gamma^W_{r_i} w^W_{r_i} + \gamma^R_{r_i} w^R_{r_i} + \gamma^E_{r_i} w^E_{r_i} + \gamma^O_{r_i} w^O_{r_i} + \gamma^N_{r_i} w^N_{r_i}\} + \sum_{r=1}^{v} \sum_l \delta_{r_l} w_{r_l} \quad (5.4.16)$$

$$S^I_{r_i} + \varepsilon^I_{r_i} - \eta^I_{r_i} = G^I_{r_i} \quad \cdots\cdots (5.4.19)$$

$$S^I_{r_i} \geqq g^I_{r_i} \quad \cdots\cdots (5.4.20)$$

$$\frac{\varepsilon^C}{\lambda^C} = \frac{\varepsilon^I_{r_i}}{\lambda^I_{r_i}} \quad \cdots\cdots (5.4.21)$$

$$S^D_{r_i} + \varepsilon^D_{r_i} - \eta^D_{r_i} = G^D_{r_i} \quad \cdots\cdots (5.4.22)$$

$$S^D_{r_i} \geqq g^D_{r_i} \quad \cdots\cdots (5.4.23)$$

$$\frac{\varepsilon^C}{\lambda^C} = \frac{\varepsilon^D_{r_i}}{\lambda^D_{r_i}} \quad \cdots\cdots (5.4.24)$$

<u>Nonlinear Goal Constraints</u>

For $r = 1, \cdots\cdots, v$; $i = 1, \cdots\cdots, m_r$ the following constraints hold :

$$\frac{1}{Q^L_{r_i}} \{ B^L_{r_i*}(Q^L_{r_i*} - x^I_{r_i} - x^D_{r_i} - \sum_{s \in z_{r_i}} y^T_{r_i s_j}) + b^3_{r_i} q^3_{r_i} + b^E w^E_{r_i} + b^O w^O_{r_i} + B^L_{s_j*} y^T_{s_j r_i} \} - \varepsilon^B_{r_i} + \eta^B_{r_i} = G^B_{r_i} \quad (5.4.25)$$

$$\frac{1}{Q^L_{r_i}} \{ B^L_{r_i*}(Q^L_{r_i*} - x^I_{r_i} - x^D_{r_i} - \sum_{s \in z_{r_i}} y^T_{r_i s_j}) + b^3_{r_i} q^3_{r_i} + b^E w^E_{r_i} + b^O w^O_{r_i} + B^L_{s_j*} y^T_{s_j r_i} \} \leqq g^B_{r_i} \quad \cdots\cdots (5.4.26)$$

$$\frac{\varepsilon^C}{\lambda^C} = \frac{\varepsilon^B_{r_i}}{\lambda^B_{r_i}} \quad \cdots\cdots (5.4.27)$$

$$Q^U_{r_i} = Q^L_{r_i*} \quad \cdots\cdots (5.4.12)$$

$$B^U_{r_i} = B^L_{r_i*} \quad \cdots\cdots (5.4.13)$$

$$B^T_{s_j r_i} = B^L_{s_j*} \quad \cdots\cdots (5.4.14)$$

5.5 Nonlinear Goal Programming Based on Cutting Plane Method

In what follows we shall develop a modified goal programming which bases its solution finding process on the cutting plane method developed by Kelley. Before delving into the details of this method, let us briefly examine our original goal programming in terms of geometry.

5.5.1 Geometrical Interpretation of Goal Programming

Basic linear algebra shows that as graphically depicted in Figure 5.5.1 the distance d between a given point x_0 and the foot of the perpendicular drawn from it onto a given hyperplace ($'ax = c$, say) is so calculated

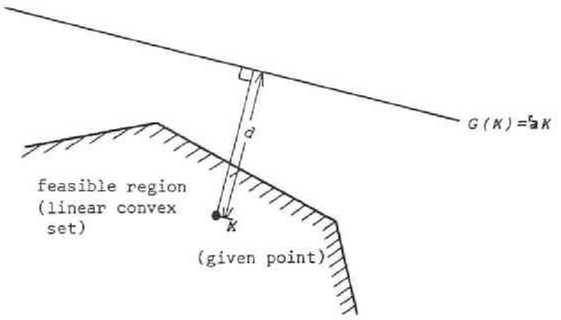


Fig. 5.5.1 Geometry of Perpendicular to a Linear Hyperplane

$$d=\frac{{}^t\boldsymbol{a}\boldsymbol{x}-G}{\sqrt{{}^t\boldsymbol{a}\cdot\boldsymbol{a}}} \quad \cdots\cdots (5.5.1)$$

By noting that any goal constraint concerned with its satisfied-level can be expressed as

$$ {}^t\boldsymbol{a}\boldsymbol{x}+\varepsilon-\eta=G \quad , \quad \cdots\cdots (5.5.2)$$

the above equation can be converted to the following:

$$d=p(\varepsilon-\eta) \quad \cdots\cdots (5.5.3)$$

where

$$p=\frac{1}{\sqrt{{}^t\boldsymbol{a}\cdot\boldsymbol{a}}} \text{ (constant)}. \quad \cdots\cdots (5.5.4)$$

This means to imply that the magnitude of $y-z$ is in proportion to the distance between a given point and the foot of the perpendicular drawn from it onto the hyperplane representing the satisfied-level condition of a given goal. Stated otherwise, our original goal programming approach can be interpreted as a mathematical expression of the problem to find a point as close to the goal hyperplane as possible subject to the technical and physical constraints. This is, however, somewhat misleading, because our objective function is so formulated that ε be minimized, but $y-z$ not necessarily being minimized. This means that a decrease in η which might be geometrically interpreted as a step further close toward the hyperplane, would never contribute to the improvement of our solution. Accordingly it needs to be added that the solution is limited to some neighborhood domain of the set of points on the goal vectors if the unbiased attainments of all the goals are pursued.

Now that it becomes evident that the deviational variables y and z are geometrically interpreted as such, our next concern is to develop a modified approach in which this geometric property would never be lost.

5.5.2 Modified Approach by Cutting Plane Method

The difficulty involved in uniquely determining the hyperplane with respect to the satisfied-level condition can be overcomed in a manner that follows.

Let us first define our goal programming problem in a general form as

$$\text{Minimize} \quad Z=\varepsilon_0 \quad \cdots\cdots (5.5.5)$$

$$A\boldsymbol{x}+B(\varepsilon_0-\eta_0)=\boldsymbol{b} \quad \cdots\cdots (5.5.6)$$

$$G(\boldsymbol{x})+\varepsilon_0-\eta_0=G \quad \cdots\cdots (5.5.7)$$

$$G(\boldsymbol{x})\geqq g \quad \cdots\cdots (5.5.8)$$

$$\boldsymbol{x},\boldsymbol{\varepsilon},\boldsymbol{\eta}\geqq 0,\ \boldsymbol{\varepsilon}=(\varepsilon_0,\boldsymbol{\varepsilon}_0),\ \boldsymbol{\eta}=(\eta_0,\boldsymbol{\eta}_0) \quad \cdots\cdots (5.5.9)$$

where $G(\boldsymbol{x})$ is a nonlinear function and Equations (5.5.7) and (5.5.8) represent the nonlinear goal constraints in question. Equation (5.5.6) represents technical and physical constraints as well as the other goal constraints expressed in linear forms.

Let $\boldsymbol{x}^{(1)}$ be a given point defined on the compact polyhedral convex set $S=\{\boldsymbol{x}|A\boldsymbol{x}\leqq\boldsymbol{b}\}$ and let $R=\{\boldsymbol{x}|g\leqq G(\boldsymbol{x})\leqq G\}$ be nonempty and $R\subsetneqq S$.

Then the hyperplane involved can be so determined that it is a tangent hyperplane to the curve of $G(\boldsymbol{x})=G$ at a certain point $\bar{\boldsymbol{x}}^{(1)}$ and that it is also in parallel with the tangent hyperplane to the graph $G(\boldsymbol{x})$ at the point $\boldsymbol{x}^{(1)}$. These hyperplanes and their points of contact are graphically illustrated as shown in Figure 5.5.2. Accordingly the distance between a given point $\boldsymbol{x}^{(1)}$ and the foot of the perpendicular drawn from it to the hyperplane with respect to the nonlinear goal constraint can be uniquely determined as also shown in Figure 5.5.2. Therefore our modified goal programming problem can be geometrically interpreted as a mathematical problem of finding a point as close to the set hyperplane as possible. This can also he expressed in terms of algebra as:

Find $(\boldsymbol{x}^{(k)}, \varepsilon^{(k)}, \eta^{(k)})$ such that it minimizes

$$z = \varepsilon_0 \tag{5.5.5}$$

subject to

$$A\boldsymbol{x} + B(\varepsilon_0 - \eta_0) = \boldsymbol{b} \tag{5.5.6}$$

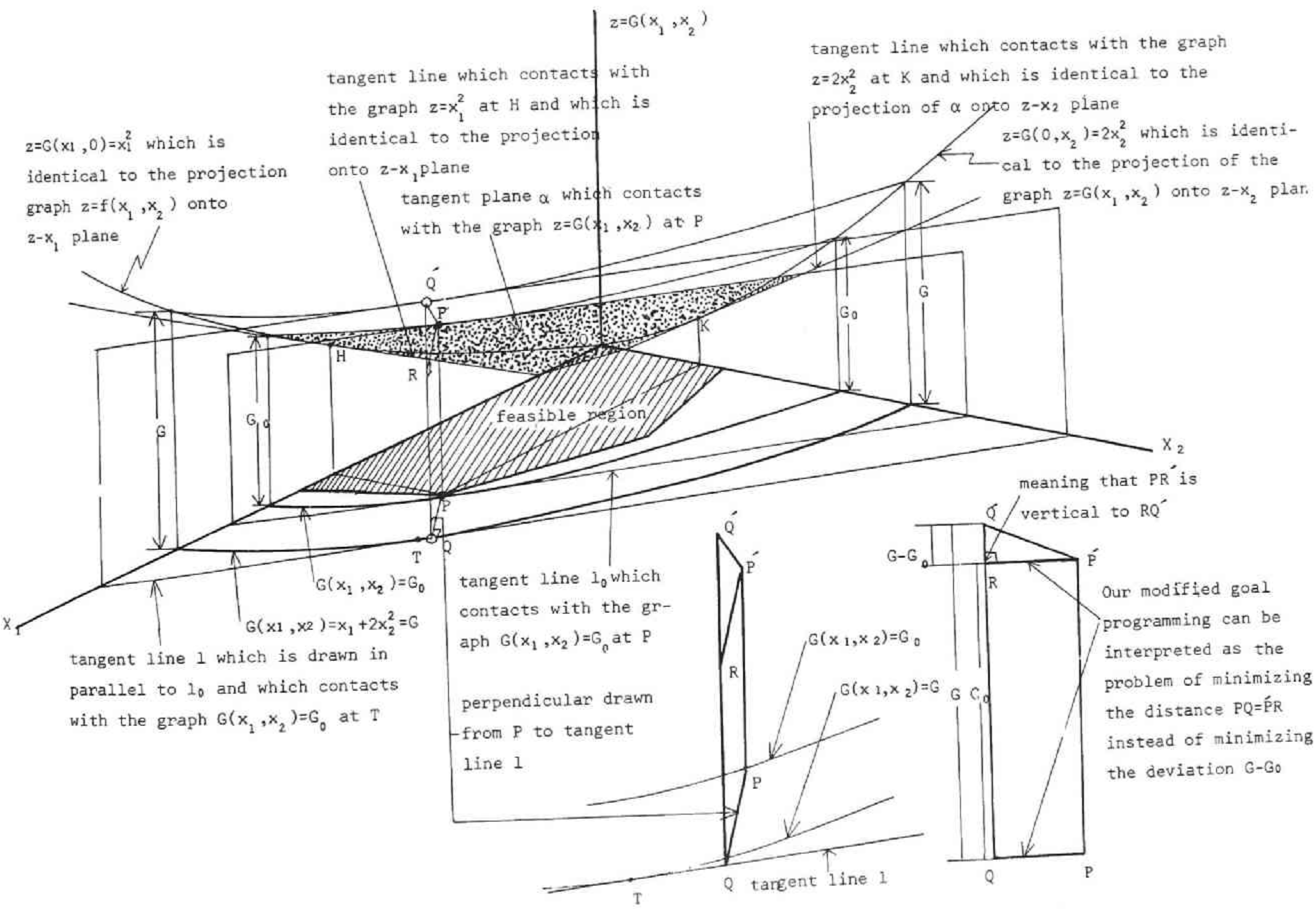


Fig. 5.5.2 Illustrated Geometry of Nonlinear Goal Programming

$$G(x^{(k-1)}) + \nabla G(x^{(k-1)})(x - x^{(k-1)}) + \varepsilon_o - \eta_o = G \tag{5.5.10}$$

$$G(x^{(k-1)}) + \nabla G(x^{(k-1)})(x - x^{(k-1)}) \leqq G \tag{5.5.11}$$

$$x, \varepsilon, \eta \geqq 0 \ , \ \varepsilon = (\varepsilon_o, \varepsilon_o), \ \eta = (\eta_o, \eta_o) \tag{5.5.9}$$

where the supercript (k) denotes the number of iterations being tried on.

This procedure terminates if and only if $x^{(k)}$ and $x^{(k-1)}$ satisfy the following condition:

$$\left| x^{(k)} - x^{(k-1)} \right| < \varepsilon \tag{5.5.12}$$

where ε is a properly small value set a priori.

There are many cases where this procedure converges to the exact solution. This is especially true when the nonlinear function $G(x)$ posseses a relatively small curvature in the neighborhood of the exact solution and convexity of the function is warranted. (See Figures 5.5.3 and 5.5.4.)

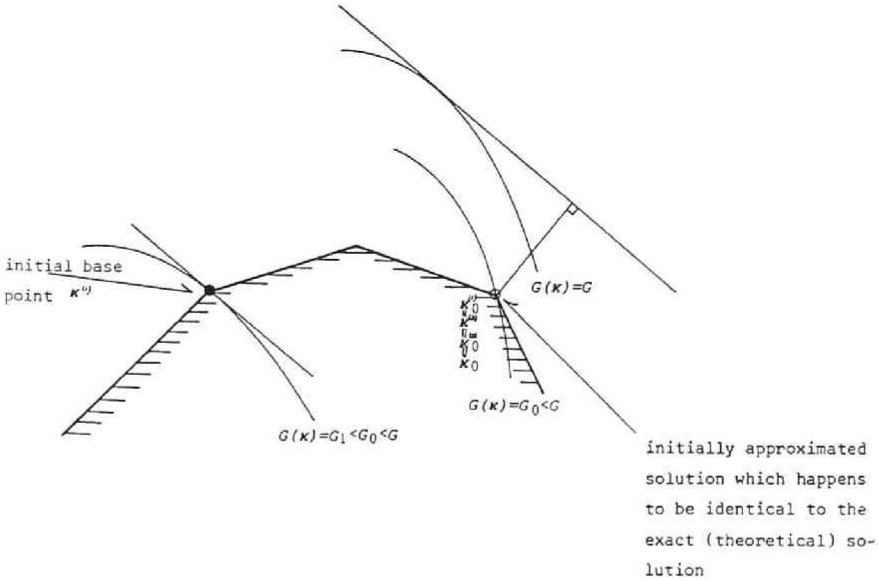


Fig. 5.5.3 Example of Valid Solution (1)

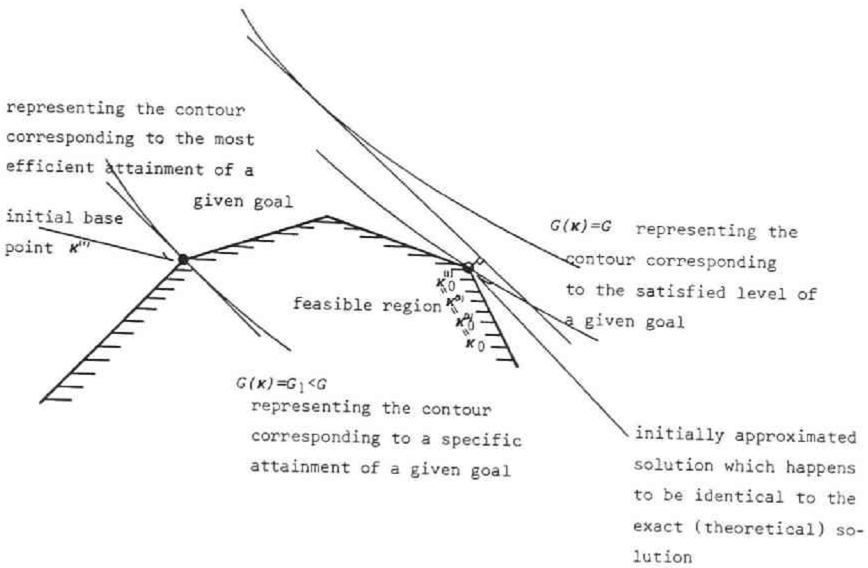
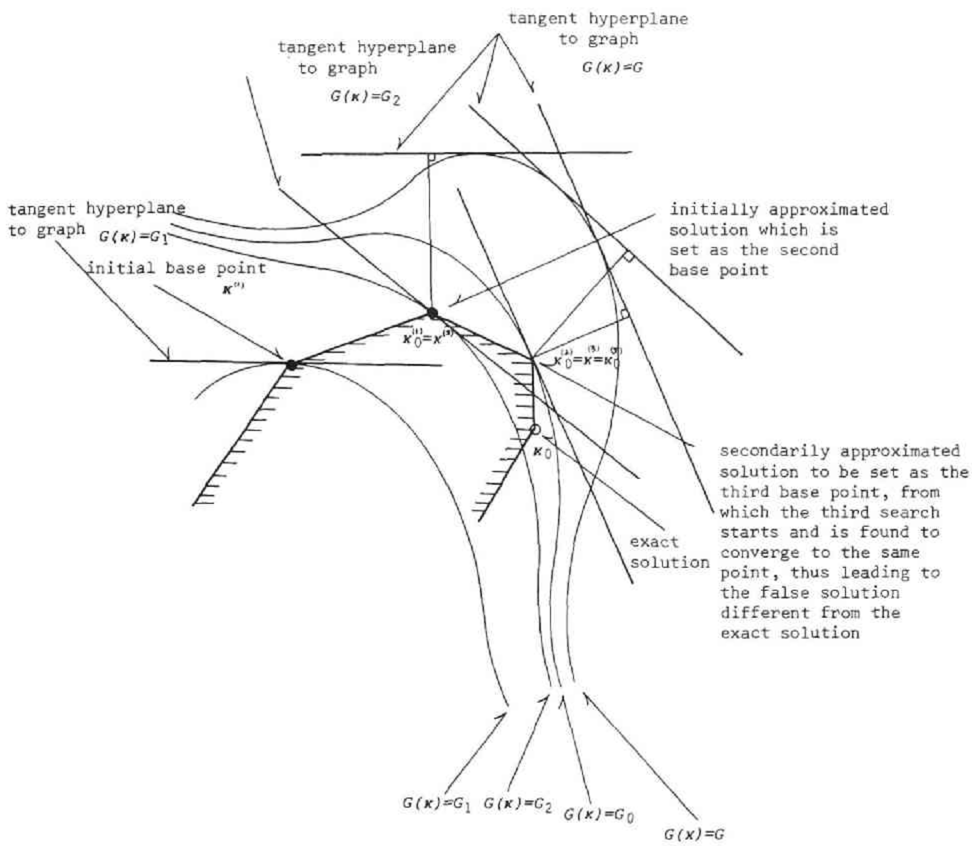


Fig. 5.5.4 Example of Valid Solution (2)

But it must also be noted that the above procedure will not necessarily lead to the exact solution as illustrated in Figure 5.5.5. In this regard the following discussions seem to be of vital importance.

(i) In case $G(x)$ is convex, there is no guarantee of achieving the desired global optimum point (the exact solution). Since the function $G(x)$ is not known to be convex, our procedure is to begin the search at a number of initial base points widely separated; and if the same value is found for all test cases, this value can be termed global



$G(\kappa)=G$: contour corresponding to the satisfied level of a given goal

$G(\kappa)=G_0$: contour corresponding to the most efficient attainment level

$G(\kappa)=G_2$: contour corresponding to the erroneously-calculated attainment level

$G(\kappa)=G_1$ contour passing the initial base point

$G > G_0 > G_2 > G_1$ in case the attainment of the goal is to be maximized

$G < G_0 < G_2 < G_1$ in case the attainment is to be minimized

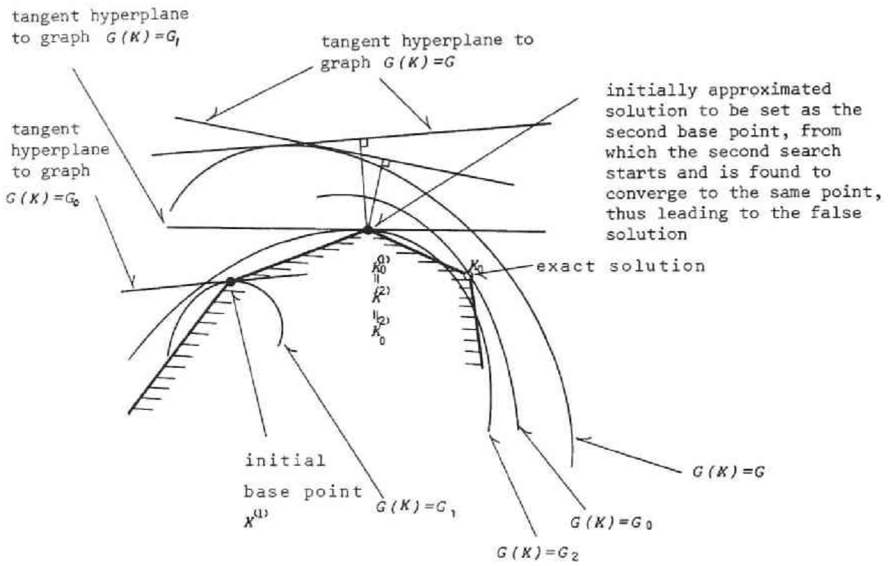
Fig. 5.5.5 Example of Invalid Solution (1)

with some degree of confidence. If several optima are found, the best, of course, is selected.

(ii) Even if $G(\boldsymbol{x})$ is known to be convex, difficulties can possibly arise for those cases where the function $G(\boldsymbol{x})=G$ forms a strictly curving boundary as shown in Figure 5.5.6. In these cases also we need to start from a new base location and again looking for another peak.

5.6 Case Study on Southern Part of Hyogo Prefecture

In likewise as our preceding studies of Chapters 2 and 4, the southern part of Hyogo Prefecture was selected as our study area where the five major streams (the Chigusa, Ibogawa, Yumesaki, Ichikawa, and Kakogawa Rivers) run in parallel with one another from the northern hilly countries down through the southern flat countries, all flowing into the Seto Inland Sea.



$G(K)=G$	contour corresponding to the satisfied level of a given goal
$G(K)=G_0$.	contour corresponding to the most efficient attainment level
$G(K)=G_2$	contour corresponding to the erroneously-calculated attainment level
$G(K)=G_1$.	contour passing the initial base point

Fig. 5.5.6 Example of Invalid Solution (2)

5.6.1 Model Data

1) Zoning

Taking account of the differences in water uses, and hydrological conditions as well as institutional and economical boundaries, the zoning of the study area was preplanned as shown in Fiugre 5.6.1.

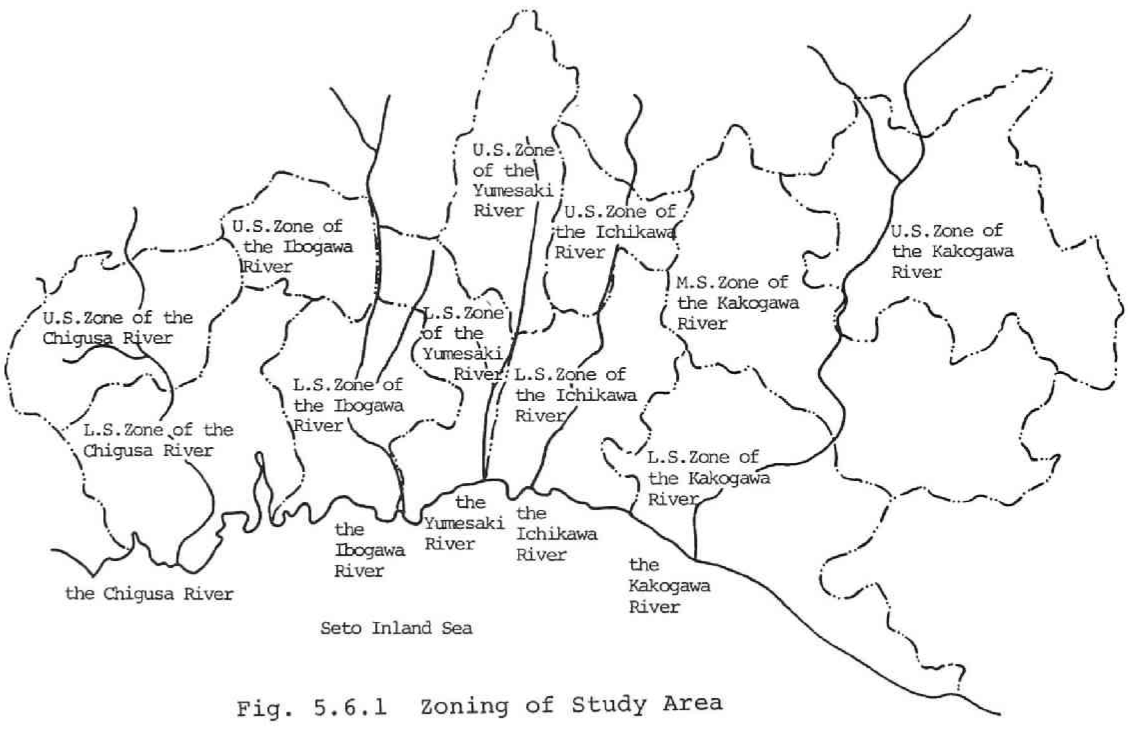


Fig. 5.6.1 Zoning of Study Area

2) Projected Water Demands

By setting each zone as a geographical unit for predicting water demands and forecasting the demands per capita and those per unit of production at the time of 1985, 10 years (being set as the planning time horizon) ahead of 1975 (being set as the outset of the project implementation) as well as the increased population and production by that time for each zone, the projected water demands were estimated as tabulated in Table 5.6.1.

3) Estimated River Discharges

By constructing the flow-duration curve for each river based on the data (1970-1974) provided by the authorities concerned,[12)13)14)] that value of flow was obtained such that the percent of time it is equal to or less than that value is around 20 percent. On this basis the sum of the minimal discharge requirement $q^1_{r_i}$ and the amount of existing collected water $q^2_{r_i}$ were estimated as shown in Table 5.6.2.

4) Estimated Cost Curves

Practical experience shows that many, if not most, of the cost curves at issue exhibit the mold of linearity, chiefly owing to the conventional unit-cost estimation method on the basis of which the curves were plotted against a given scale of facilities, and which seems to be only available and effective in case we have limited information on related costs. (See Table 5.6.3.) On the contrary, those cost curves for the filtration plant, wastewater and tertiary treatment plants, proves to be expressed by the nonlinear concave functions with respect to their scales as shown in Figures 5.6.2 to 5.6.3. Here again we seem to be confronted with the problem of nonlinearity.

This problem may effectively be handled by the cutting plane method as

Zone \ Demands	Projected Industrial Water Demand ($10^3 m^3$/day)	Projected Domestic Water Demand ($10^3 m^3$/day)
U.S. Zone of the Chigusa R.	3.99	1.05
L.S. Zone of the Chigusa R.	42.55	10.75
U.S. Zone of the Ibogawa R	7.95	2.07
L.S. Zone of the Ibogawa R.	72.82	16.67
U.S. Zone of the Yumesaki R.	3.89	1.22
L.S. Zone of the Yumesaki R.	40.27	9.87
U.S. Zone of the Ichikawa R.	13.75	4.65
L.S. Zone of the Ichikawa R.	161.09	39.27
U.S. Zone of the Kakogawa R.	38.14	9.72
M.S. Zone of the Kakogawa R.	76.40	19.85
L.S. Zone of the Kakogawa R.	189.78	119.40

Table 5.6.1 Projected Water Demands

Zone	River Discharge ($10^3 m^3$/day)	Water Quality Standards (BODppm)
U.S. Zone of the Chigusa R.	127	5
L.S. Zone of the Chigusa R.	258	10
U.S. Zone of the Ibogawa R.	389	5
L.S. Zone of the Ibogawa R.	400	10
U.S. Zone of Yumesaki R.	32	5
L.S. Zone of the Yumesaki R.	71	10
U.S. Zone of the Icikawa R.	183	5
L.S. Zone of the Ichikawa R.	216	10
U.S. Zone of the Kakogawa R.	483	5
M.S. Zone of the Kakogawa R.	574	6
L.S. Zone of the Kakogawa R.	752	10

Table 5.6.2 Estimated River Discharges and Established Water Quality Standards

Basins	1	2	3	4	5	6
Chigusa R.	6.51	7.83	22.00	26.63	27.69	37.33
Ibogawa R.	5.82	12.31	17.81	38.86		
Yumesaki R.	3.12	27.67				
Ichikawa R.	7.00	15.15	15.27	41.10		
Kakogawa R.	8.67	9.65	10.18	13.03	15.65	17.03

(10^3yen/m^3/day)

Table 5.6.3 Estimated Unit Costs for Different Dams

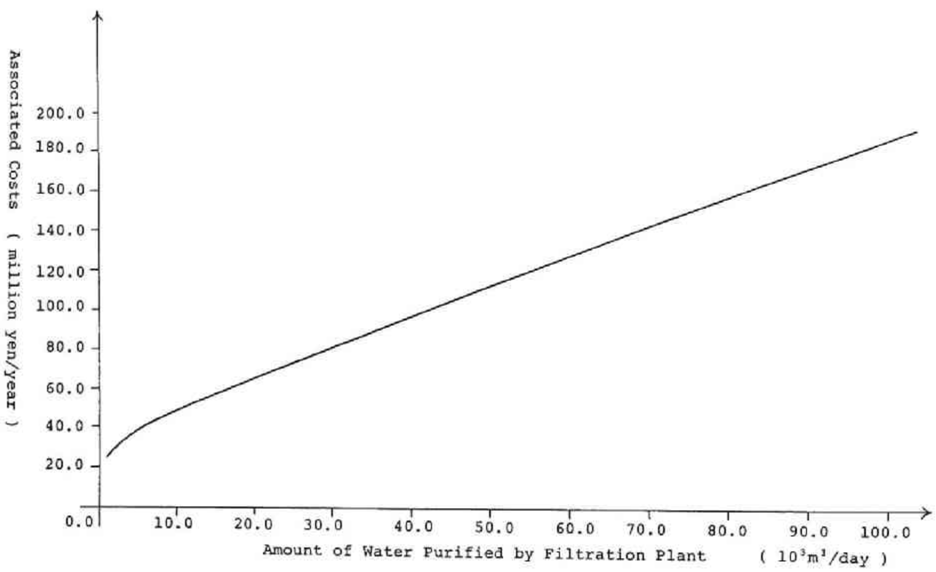


Fig. 5.6.2 Estimated Cost Curve for Filtration Facilities

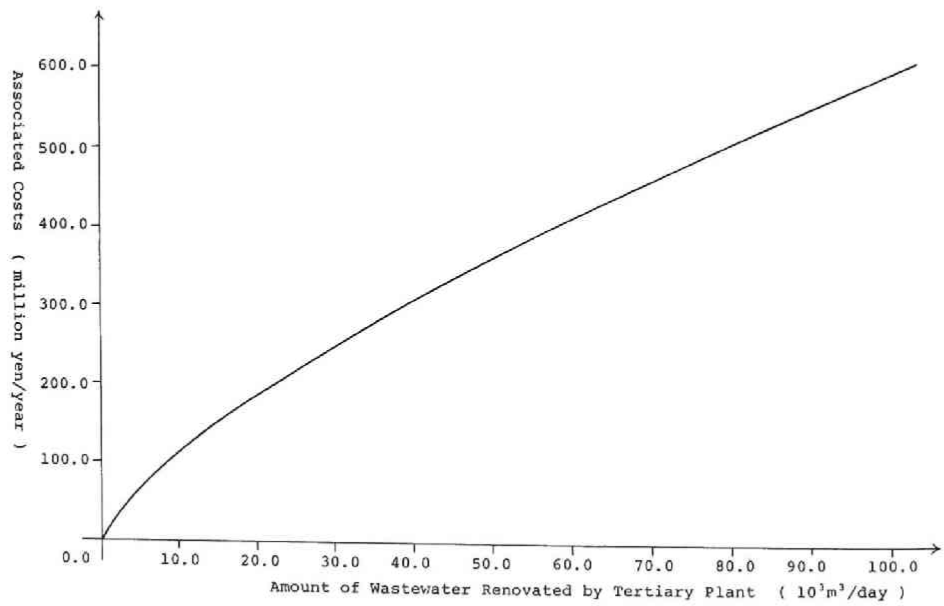


Fig. 5.6.3 Estimated Cost Curve for Tertiary Treatment Plant

utilized on our modified goal programming approach which replaces the nonlinear constraints step by step by the linearized ones. Hence, this approach will be attempted to handle the said nonlinear constraints.

5) Water Quality Standards

By taking account of the averages of the BOD measurements in the effluents from common wastewater plants and tertiary treatment plants, the values for b^E and b^0 were taken to be 20 ppm and 4 ppm, respectively. The BOD's in the streamflows of the tributary rivers were set to be equal to 2ppm by assuming that they are so regulated to meet the standard values officially established by the Prefectural Government of Hyogo. The BOD standards for the major rivers were prescribed in a likewise manner. This explanation immediately follows.

5.6.2 Calculation Cases

Before performing our calculations the following cases were preselected.

(i) All the cases are classified into three types, i.e., Cases A, B and C which are characterized by relatively high quality standards, medium ones and lower ones, respectively.

(ii) Case A is further subcategorized as Cases A-(1), A-(2) and A-(3) according to the difference in the values of the permitted-levels of the supply-goals. Likewise Cases B and C are further categorized as Cases B-(1), C-(1); B-(2), C-(2); and B-(3) and C-(3). It might well be interpreted that Cases A-(1), B-(1) and C-(1) are the problems of selecting "demand-regulatory alternatives", Cases A-(2), B-(2) and C-(2) those of finding "medium alternatives", and Cases A-(3), B-(3) and C-(3) those of choosing "demand-dependent alternatives". (See Table 5.6.4.)

Goals	Industrial-supply-goal ($10^3 m^3$/day)				Domestic-supply-goal ($10^3 m^3$/day)				Quality-goal (BOD ppm)			
Cases / Zones	$G^I_{r_i}$	$g^I_{r_i}$ (1)	(2)	(3)	$G^D_{r_i}$	$g^D_{r_i}$ (1)	(2)	(3)	$g^B_{r_i}$	$G^B_{r_i}$ A	B	C
U.S.Zone of the Chigusa R.	1.05	0.84	0.94	1.00	3.99	3.19	3.59	3.79	5	4	3	2
L.S.Zone of the Chigusa R.	10.75	8.60	9.68	10.20	42.55	34.04	38.30	40.42	10	9	8	7
U.S.Zone of the Ibogawa R.	2.07	1.66	1.86	1.97	7.95	6.36	7.16	7.55	5	4	3	2
L.S.Zone of the Ibogawa R.	16.67	13.34	15.00	15.80	72.82	58.24	65.52	69.18	10	9	8	7
U.S.Zone of the Yumesaki R.	1.22	0.98	1.10	1.16	3.97	3.18	3.57	3.79	5	4	3	2
L.S.Zone of the Yumesaki R.	9.87	7.86	8.84	9.38	40.27	32.22	36.24	38.26	10	9	8	7
U.S.Zone of the Ichikawa R.	4.65	3.72	4.19	4.42	13.75	11.00	12.38	13.06	5	4	3	2
L.S.Zone of the Ichikawa R.	39.27	31.42	35.34	37.31	161.09	128.87	144.98	153.04	10	9	8	7
U.S.Zone of the Kakogawa R.	9.72	7.80	8.72	9.23	38.14	30.51	34.33	36.23	5	4	3	2
M.S.Zone of the Kakogawa R.	19.85	15.88	17.87	18.86	76.40	61.12	68.76	72.58	6	5	4	3
L.S.Zone of the Kakogawa R.	119.40	95.52	107.46	113.40	189.78	151.82	170.80	180.30	10	9	8	7

Table 5.6.4 Calculation Cases

5.6.3 Calculation Results

To begin, we shall take Case A-(1) as the standard case and make an in-depth analysis of its results.

1) Standard Case (Case A-(1))

The results for this case are diagrammatically shown in Figure 5.6.4, from which the following may readily be understood.

(i) A relatively small amount of river flow is diverted from the upperstream of the Ichikawa River to both the upperstream and midstream of the Kakogawa River.

(ii) Instead of inter-basin streamflow diversion systems, another type of inter-basin water transfers, that is, inter-zonal industrial water distribution systems are also implemented from the downstream zone of the Ichikawa River to that of the Yumesaki River where 38 percent of those distributed are provided for its own use there, and 4 percent for its nearest upperstream zone, and from there to the downstream of the Ichikawa River the remaining 58 percent are distributed.

(iii) In the basin of the Chigusa River it seems better to establish an intra-basin system which is separated from the other basins.

(iv) In each basin some of those purified industrial waters are distributed upward to its upperstream zone.

(v) So far as the reclamation system is concerned, its implementation is concentrated on the downstream zones of the Ichikawa and Kakogawa Rivers, from either of which 21 percent of the renovated waters are further distributed to

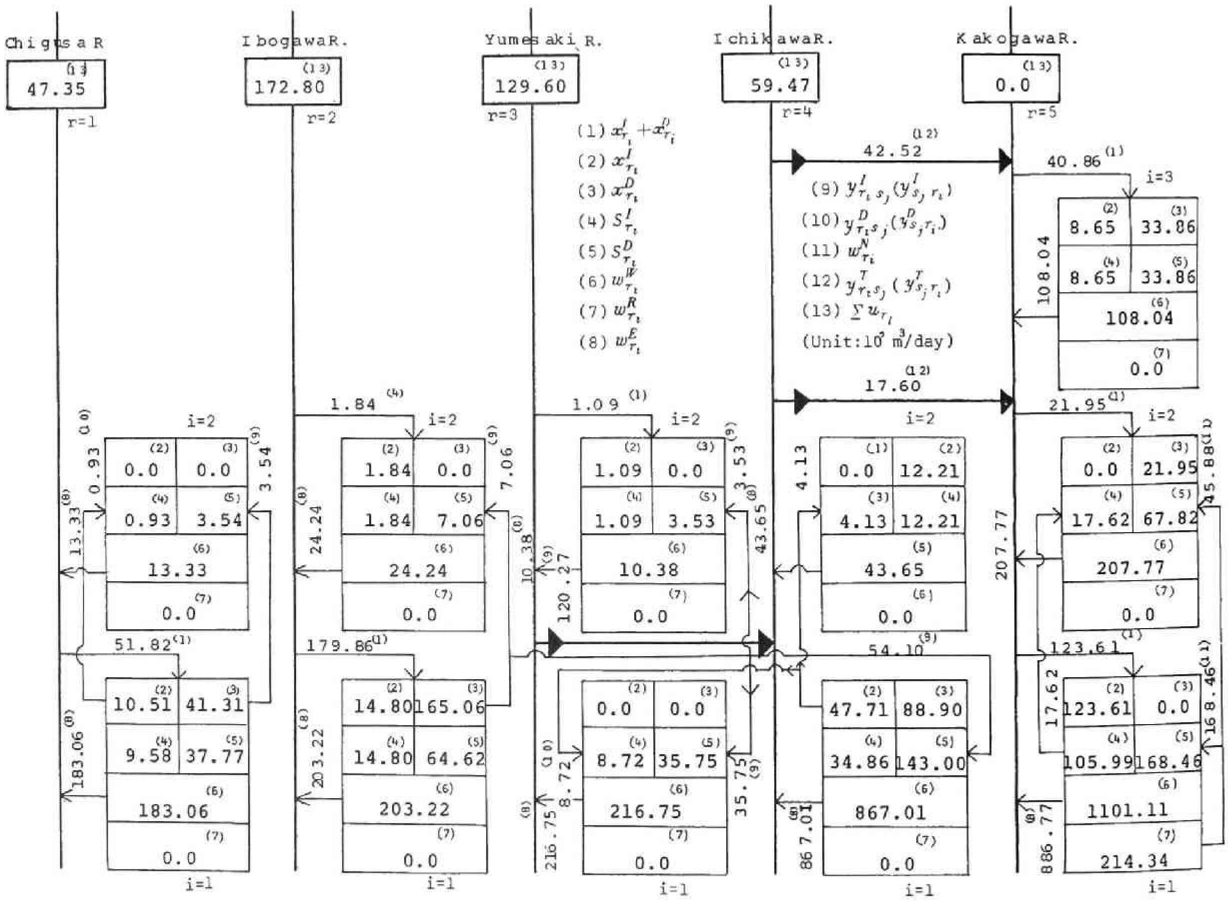


Fig. 5.6.4 Calculation Results for Standard Case

their upward zones to supplement the provisions of water for industrial uses there.

One might expect that the distribution system would be more economically implemented so as to convey water from upperstream (midstream) zones to their downstream zones than to convey vice versa. We shall examine this point later.

2) In-depth Analysis of the Findings of Standard Case

On the basis of the above findings 1)-(i) through 1)-(v), let us carry out an in-depth analysis with an aim to probe into the underlying mechanism operating to yield those results. Once this is done, we shall be able to obtain some basic yardsticks with which to formulate our water resources management policies.

① Taking account of scales of economics related to the facilities of purification, wastewater treatment and tertiary treatment and in view that inter-basin river-flow diversions are not as much economical as inter-zonal distributions chiefly owing to a "round-about" way of supply involved in the former as compared to the latter, one might intuitively expect that if economic efficiency alone is pursued to its full extent, inter-basin water transfers, if necessary, will take the form of inter-zonal purified-water distributions rather than that of inter-basin streamflow diversions. The findings of 1)-(i) through 1)-(iii), however, show that the results derived from our model come out a little bit against our expectation, because among those transported from one basin to another totalling 316,000 m^3/day (102,000m^3/day in net quantity), only 43 percent (36 percent) are covered by the inter-zonal distribution system, and the remaining 57 percent (64 percent) by diversion channels.

② In this connection it seems evident that the implementation of the diversion system would be more effective to this goal than that of the distribution system if one observes that the former alternative augments directly the stream discharge with much less chance of degrading streamflow quality than the latter that contributes to the augmentation of streamflow only in an indirect manner and merely after waters are used and polluted whether they undergo secondary or tertiary treatments.

In light of these considerations it might be concluded that to the end of the cost-goal our alternative derived from our model proved to be a second-best policy, because a more extended scale of the inter-zonal distribution system would be more economical. Likewise we might well say that in the light of the quality-goals our alternative is also a second-best policy in the sense that higher quality of fresh-water could be more extensively developed in the upperstream valleys.

③ Next we proceed to the analysis of the findings of 1)-(v). As referred to before, it seems quite natural to expect that any form of conveyance from an upperstream (or midstream) down to its downstream would be more inexpensive than that from a downstream up to its upperstream (or midstream). This also implies that the alternative obtained from our model is a second-best policy in the light of the cost-goal and that it is also a second-best policy to the end of attaining the quality-goals.

④ Next we should study how and how much our remaining goals, i.e., supply-goals which have been put aside from our foregoing discussions, influenced the calculation results. For this purpose it seems better to assume that case where the supply-goals are excluded and only their permitted-levels set as technical constraints. Then one might readily understand that the amounts of water supplied for use in the downstream zone on each river would be reduced to their permitted-levels in order to augment the stream dicharges, because it would lead to the increased attainments of both the cost- and quality-goals.

5 From what have been studied above it will be obvious that our alternative obtained from our model can be conceived as an efficient solution, because a further improvement in the attainment of any one kind of goals would be achievable only at the expense of the others.

6 This efficient solution can be characterized by its well-balanced attainments of all the goals, mainly because of the stipulated L-type utility curves. The attainment ratio for each goal which is defined as the ratio of the deviation between the attained- and permitted-levels to that between the satisfied- and permitted-levels, is calculated as 0.50 to 0.40 for the different goals.

7 Our modified (nonlinear) goal programming approach which was developed by the authors proved to yield a very reasonable solution after a couple of iterations. The nonconvexities involved in our nonlinear goal constraints can be overcomed if such an initial base point is predetermined on the basis of the foregoing "path-breaking" computations on the model.

3) Comparative Study

(i) Let us first compare those results of Cases A-(1), A-(2) and A-(3) which are characterized by passive, ordinary and active actions, respectively, toward the conservation of water quality. Furthemore our discussions will be limited to the Kakogawa River which seems to provide us with the most suggestive information for the policy-formulations. From Figure 5.6.5 it is obvious that 1 the varying pattern of the amounts of the renovated waters is found to take a dissymmetrical form as compared to that of the diverted streamflows; 2 the augmentation of stream discharges by promoting the inter-basin streamflow diversion systems seems to be effective for the purpose of quality alleviations to certain extent, but a higher standard of water quality is effectively attainable by chiefly promoting the reclamation systems. 3 The same discussion applies to Cases B and C.

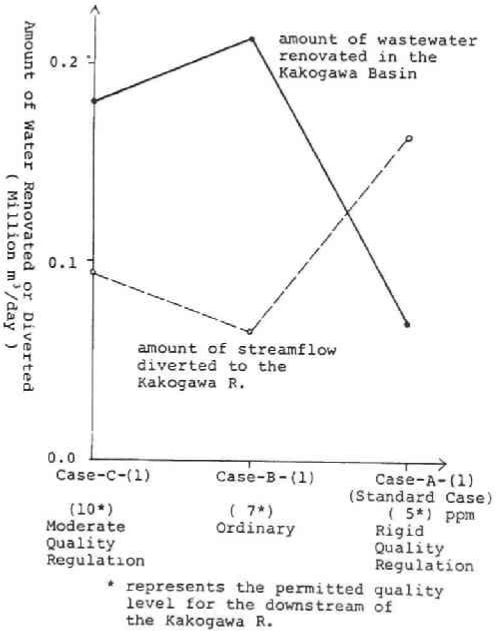


Fig. 5.6.5 Comparative Study of Cases A-(1), B-(1) and C-(1)

(ii) We shall next take Cases A-(1), A-(2) and A-(3) to analyze the results by comparison. Let us first observe that as referred to in the setting of our cases, Cases A-(1), A-(2) and A-(3) might well be called "demand-regalatory alternative", "medium alternative" and "demand-dependent alternative", respectively. The comparison of the results shown in Figure 5.6.6 reveals that if one takes a demand-dependent alternative system, a balanced-

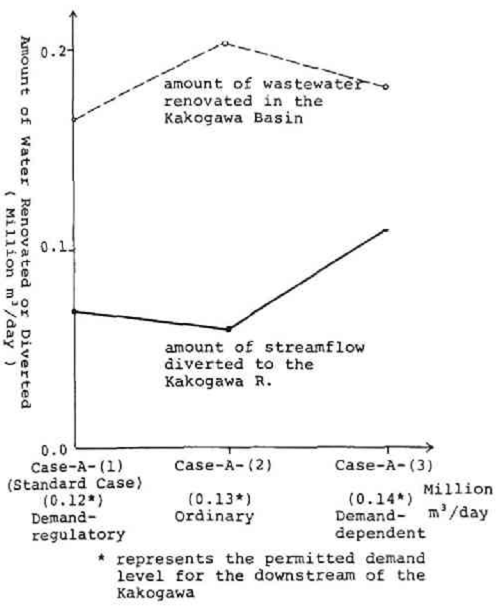


Fig. 5.6.6 Comparative Study of Cases A-(1), A-(2) and A-(3)

mix of the two modes of utilizations is most propitious, whereas if one seeks for an ordinary alternative, main sources of supplies are shared by dams and renovated waters play solely a support's part in providing water. And for a demand regulatory alternative system a little more increase in the amounts of renovated waters with a little more decrease in the amounts of fresh waters seems to be most propitious.

In this regard attention needs to be paid to the fact that the varying patterns of the amounts of fresh water and renovated water appear to be similar to that discussed in (i). This is quite natural, because if the cost-goal and supply-goals are required to retain given attainment levels, an increment in water demands would inevitably lead to such deteriorated quality that would violate our quality standards. This means that the attainment levels of the goals should be lowered until the quality restrictions are satisfied. That seems to tell us again that the variation in water quality determines the characteristics of the alternative to be obtained.

5.7 Conclusion

The central question to which this paper is addressed is the coordination problem of the attainments of the different kinds of goals assigned to the planning of the water resources systems of inter-basin development and dual-modal utilization. At the outset of our study we placed into perspective those different goals involved in the planning of this field and gave primary consideration to the specification of those goals.

We shall summarize here our findings of this chapter.

(i) In the basin of the Chigusa River it seems better to establish an intra-basin system which is separated from the other basins.

(ii) So far as the Kakogawa River is concerned and if the water quality is confined to be less than 3ppm in the upperstream, 5 to 6 ppm in the midstream and 7 to 10 ppm in the downstream, it appears that the inter-basin streamflow diversion system should be implemented from the Yumesaki River via the Ichikawa River to the downstream basin of the Kakogawa River so as to meet both the demands and the given standards of quality.

But things turn out to be somewhat different if one wishes to achieve a higher quality management especially in the upperstream and midstream. That is to say that a relatively small amount of streamflow is diverted from the upperstream of the Ichikawa River to both the upperstream and midstream of the Kakogawa River. On the contrary the diversion is not implemented from the downstream of the Yumesaki River via the Ichikawa to that of the Kakogawa. Instead, the reclamation system needs to be implemented on a larger scale. This seems to be derived from the fact that the increased necessity for augmenting streamflow might be more efficiently meeted by extending the scale of the reclamation system than increasing the amounts of diverted water or developing more fresh water on its upperstream valley.

(iii) So far as the Ichikawa River is concerned, the implementation of the inter-basin river flow diversion system as well as the inter-zonal industrial water distribution system seems to be consistently required irrespective of the difference in the water quality management.

(iv) Anyhow the inter-basin water transfer systems are believed to be necessary insofar as the Ichikawa and Kakogawa Rivers are concerned. The diversion system should be introduced primarily for the purpose of improving streamflow quality rather than to the end of water supplies. On the other hand the implementation of the inter-zonal distribution system seems to be most adequate for securing increased amount of water supply but not propitious to the conservation of streamflow quality.

(v) The modified goal programming approach developed by the authors proved to serve as an effective tool for handling nonlinear goal constraints which are frequently encountered in many of practical problems.

From this broad base of information a narrowed set of recommendations could be developed after appropriate economic and technical studies were subsequently carried out. Some of them are:

① How high and in what specific a manner should we set the levels of the goals to aim for?

② Is it an acceptable assumption to the planner of the practical field of water resources that all the goals should be well-achieved?

③ Is it reasonable to treat water quality with a single parameter, i.e., BOD and should we not consider more detailed mechanism of water quality changes?

④ The uncertainties involved in estimating costs, the rate and direction of technological innovation should be given more detailed attention.

For all those questions to be further analyzed in future, it seems to be important to go as far as possible with the model presented in this paper and it is believed by the authors that enough information would come from this kind of analysis.

References

1) Okada, N., and Yoshikawa, K.: Nonlinear Goal Programming Approach to Inter-basin, Multi-modal Water Assignment Problem, Proc. of JSCE (To Be Issued).

2) Okada, N., and Yoshikawa, K.: Modelling Multi-goal, Inter-basin, Surface Water Development System and Analysis of Supplementary Utilization of Reclamation System, Memoirs of the Faculty of Engineering, Kyoto University (To Be Issued).

3) Sakurai, H.: Systems Analysis of Multi-goal, Inter-basin, Multi-modal Water Assignment Problems, Master Thesis Presented to the Faculty of Engineering, Kyoto University, March, 1976 (In Japanese).

4) Yagi, Y.: Goal Programming Model of Multi-goal, Inter-basin, Water Utilization System, Graduation Thesis Presented to the Faculty of Engineering, Kyoto University, March, 1976 (In Japanese).
5) Okada, N., and Yoshikawa, K.: Nonlinear Programming Approach to the Water Assignment Problem for a Single River Basin, Proc. of JSCE, No.247, March, 1976 (In Japanese).
6) Charnes, A., Cooper W.W., Devoe, J.K., Learner, D.B., and Reinecke, W.: A Goal Programming Model for Media Planning, Management Science, Vol.14, No.8, 1968.
7) Lee, S.M.: Goal Programming for Decision Analysis, Mason and Lipscomb Publ., 1972.
8) Ruefli, T.W.: A Generalized Goal Decomposition Model, Management Science 17, No.8, 1971.
9) Ijiri, Y.: Management Goals and Accounting for Control, North-Holland Publ. Comp., 1965.
10) Fushimi, T., and Yamaguchi, T.: Mathematical Methodology for Finding Balanced Attainments of Multiple Goals, Journal of the Operations Research Society of Japan, Vol.19, No.2, 1975 (In Japanese).
11) Kelley, J.E.: The Cutting Plane Mehtod for Solving Convex Programs, Journal of the Society for Industrial and Applied Mathematics, 8, 1960.
12) Ministry of Construction: Annual Bulletin of Stream Discharge, 1971-1974 (In Japanese).
13) Ministry of Construction: Annual Bulletin of Stream Quality, 1974 (In Japanese).
14) Hyogo Prefecture: Data on Discharge and Quality of Main Streams of Hyogo, Unpublished Material, 1974 (In Japanese).

Chapter 6 Nonlinear Programming Approach for Analysis and Design of Water Distribution System

6.1 Introduction

Two topics to be covered in both this chapter and the subsequent one (Chapter 7) concern the water supply and distribution system, which is considered a subsystem of the water resources utilization system, that is, a subsystem related to the provision of water at the extremities of the total system. The specified feature of the said subsystem to which explicit consideration will be given in this chapter is the optimal implementation of the distribution system,[1)2)3)4)] whereas the treatment in the subsequent chapter places a specified focus on the optimal operational control of the water supply and use system in the phase of droughts.

6.2 Water Distribution System

Extensive distribution systems are needed to deliver water to the individual consumer in the required quantity and under a satisfactory pressure. The design or extension of this water distribution system generally involves a large initial capital outlay as well as the continuing cost of operation, maintenance and repairs, and is often the major investment of a municipal waterworks. Therefore selection of the most economical system in the design phase should receive due concern of the engineers in this field. But because of the complex nature of the problem arising from the great number of design components and their interactions, a minimum cost design is very difficult to obtain by trial and error. Although the pursuit of optimal design is not a new idea, relatively few work in this line has already been reported by the researchers home and abroad. They possess several inherent characteristics that are discouraging : 1 Some of them lack mathematical justification of modelling and/or solution algorithms. 2 The systems treated are oversimplified, which seem to limit largely the applicability of the presented models to practical design problems. In 1966, Sueishi motivated the availability of the method of steepest ascent in minimizing the installation cost of a simple grid-iron (looping network) distribution system.[6)] Though the idea was unique and practical needs for this kind of approach claimed by him, the technique of solution search seems to demand further sophistication from a mathematical point of view. One fatal drawback to the said technique is that the treatment of inequality constraints especially in the neighborhood of the boundary conditions lacks due mathematical examination and in consequence the solution may be remote from the actual optimum.

The treatment of looping networks by nonlinear programming was also attempted by Jacobi,[7)] but the network presented was a single arrangement of two loops of seven pipes and no attempt was made on complex networks. Nakajima has approached the problem by observing its analogy to the shortest route search problem,[8)] but this stipulation seems to limit largely the domain of its applicability, because topographic conditions and predetermined configuration of the treated distribution system would not necessarily warrant the justification of the shortest-route pipes as the optimal basic skelton of the system.

Amongst many approaches already presented, those due to Watanatada and Lam are considered most advantageous;[9)10)] the former evolving an approach based on the "gradient balancing" concept of Haarhoff and Buys,[11] and the latter approaching the problem by virtue of the modified Newton Raphson Method.

6.3 Objectives of the Study

Although the study included in this chapter is motivated in part by the aforementioned work, its objectives derive from some original concerns : ① provision of basic information needed for the "optimal" design of a distribution system through operational handlings of the model to be developed, thereby identifying the notion of "optimal" with the minimum cost; ② comparative analysis of the efficiencies of the different solution algorithms developed by the authors; and ③ application of the model to some practical problems, with a view to demonstrate the potential of the model as a tool to produce some adequate alternative plans of the development of a distribution system in the phase of plan-formulation other than that of design-making, thereby specific focus being placed on the integrated implementation of multiple distribution systems related to different localities.

In light of these considerations the problems to be considered are identified as follows.

6.4 Plan of the Study

In light of above considerations a mathematical model will be constructed in 6.6. That is, 6.6 deals with the mathematical formulation of the optimal design problem where the criterion will be selected as the minimization of the total implementation costs. 6.7 discusses the developments of three different solution algorithms for the model. In 6.8 a comparative study will be conducted to identify the advantages and disadvantages of those algorithms. Then in 6.9 the model will be applied to the optimal design of a simple network system with a view to provide some basic yardsticks with which to design distribution systems. Finally 6.9 deals with the integrated development of an area-wide water distribution system with a case study on the jointed area covering parts of both Kakogawa and Takasago Cities. Thereby our focus will be placed on the potential of the model when applied to this kind of planning problem involved in its development. To conclude our discussions the obtained results are summarized with the assessment of the potential of the extended systems approach with the model or its variants.

6.5 Identification of the Problem

(i) The gridiron system (looping network system) will be given explicit consideration, the reasons being : ① another type of distribution system, that is, a treelike distribution system with many deadends is unsatisfactory because water may become stagnant at the extremities of the system; ② moreover as for the latter system, if repairs are necessary a large district must be cut off from water; ③ also in the latter system, with a locally heavy demand headloss may be excessive unless the pipes are quite large; and ④ these difficulties are minimized with the gridiron layout.

(ii) If pumping is necessary, it is assumed that it will be pumped into distribution reservoir(s) instead of water being pumped directly into closed distribution lines. This assumption is vitally necessary to circumvent the difficulties which would be posed otherwise, because it justifies the implicit treatment of pumping facilities by parameterizing the inlet piezometric pressure(s) at the supply node(s). The term "supply node" is used to refer to that node which water comes in through the supply conduit leading to the distribution reservoir(s). Additionally another kind of nodes are called "demand nodes".

(iii) Since our concern is to determine the skelton of the distribution system, only supply mains are considered excluding both auxiliary mains and minor distributors (feeder pipes).

(iv) The layout of the system is predetermined by taking account of street plan, topography, location of supply works and distribution reservoirs.

(v) Within each gridiron are assumed individual consumers of water. The estimated water demands are preassigned to each of the related nodes as illustrated in Figure 6.4.1. This demand assigned to a given node will be referred to as the "outlet flow" at a given demand node.

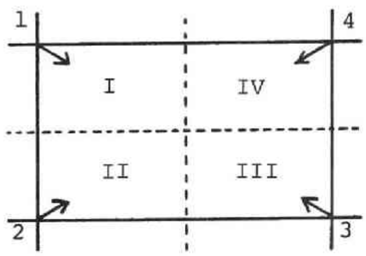


Fig. 6.4.1 Preassignment of Water Demands to Concerned Nodes

(vi) At each demand node it is required that water should be supplied in a required pressure, which will be called the "minimum allowable pressure" at each node.

(vii) The selected design variables are the pipe diameters, the pressure heads at the node discharges and head losses in pipelines. The size of the diameters is arbitrary if it falls within the range of the permissible one.

(viii) The prescribed criterion to check the optimality is considered relative whose selection depends on the primary interest specified there. We shall stand on the position that one of the most common interest in the design of the distribution system is the minimization of the cost associated with its installation and operation. Accordingly this criterion is selected throughout our study.

(ix) Other minor assumptions will be given where necessary.

6.6 Model Formulation

6.6.1 Notation

f_j : discharge in pipe j (variable)

h_j : head loss in pipe j (variable)

d_j : diameter of pipe j (variable)

l_j : length of pipe j (constant)

H_i : piezometric pressure head at node i (variable)

q_i : external discharge leaving node i ($q_i < 0$) or entering node ($q_i > 0$) (constant)

$\overline{H}_i$: minimum permissible pressure head at node i (constant)

L_i : elevation of node i (constant)

d_0 : minimum permissible pipe deameter (constant)

c : Hazen-Williams coefficient (constant)

m_1 : number of consumption nodes (constant)

m_2 : number of source nodes (constant)

m_0 : number of nodes (constant)

n : number of pipelines (constant)

p : number of loops (constant)

A : pipe-node incidence matrix, whose elements a_{ij} are $+1$ if the flow in pipe j enters node i, -1 if it leaves the node, 0 if pipe j is not incident to node i (predetermined)

6.6.2 Constraints

Kirchhoff's node law states that at each junction

$$\sum_{j \in B_i} a_{ij} f_j = q_i \quad (i=1, \cdots\cdots, m_0) \qquad (6.6.1\text{-}1)$$

hold where B_i $(i=1, \cdots\cdots, m_0)$ represent those pipelines branching from node $i\ (i=1, \cdots\cdots, m_0)$.

This can be rewritten by virtue of the incidence matrix as

$$A f = q \qquad (6.6.1\text{-}2)$$

where $f=(f_1, \cdots\cdots, f_n)$, $q=(q_1, \cdots\cdots, q_{m_0})$.

For given inflow (supply) and outflow (demand) conditions A is singular, since one node equation is redundant. Introducing a reference node, which is equivalent to omitting one node equation, yields the reduced matrix, $\overline{A}$, now nonsingular, and correspondingly $\overline{q}$, such that

$$\overline{A} f = \overline{q} \qquad (6.6.2)$$

We shall select one of the inflow node as the reference node. As a vehicle of notation this node is numbered 1. Accordingly,

$$\overline{A} = \begin{pmatrix} a_{22} & \cdots\cdots & a_{2p} \\ \vdots & & \vdots \\ a_{m_0 2} & \cdots\cdots & a_{m_0 p} \end{pmatrix} \quad \text{and} \quad \overline{q} = (q_2, \cdots\cdots, q_{m_0})$$

In each circuit Kirchhoff's loop law reads

$$\sum_{j \in C_t} h_j = 0 \qquad (6.6.3\text{-}1)$$

where C_t $(t=1, \cdots\cdots, p)$ represent those pipes consisting of the loop t.

The alternative expression of the Kirchhoff's loop law is given as

$$A^t H = h \qquad (6.6.3\text{-}2)$$

where A^t represents the transpose of the incidence matrix A, H the vector whose components are piezometric pressures at the nodes, that is $H=(H_1, \cdots\cdots, H_i, \cdots\cdots, H_{m_0})$ and $h=(h_1, \cdots\cdots, h_n)^t$.

The piezometric heads at the nodes are required to meet the minimum permissible pressure head conditions such that:

$$H_i + L_i \geq \overline{H}_i \quad (i=1, \cdots\cdots, m_0) \qquad (6.6.4)$$

The relation between the pipeflow and the head loss is given as follows.

$$h_j = c\, d_j^a\, f_j^b\, l_j \quad (j=1, \cdots\cdots, n) \qquad (6.6.5)$$

where $c=0.002$, $a=-4.87$ and $b=1.85$.

Additionally the diameters of the pipes are confined by the minimum permissible diameter conditions such that

$$d_j \geq d_0 \quad (j=1, \cdots\cdots, n) \qquad (6.6.6)$$

6.6.3 Objective Function

The objective function is taken to be the minimization of the total implementation cost. That is,

$$\text{Minimize } z=\sum_{j=1}^{n} (\alpha d_j^{\beta}+\gamma)\ l_j \qquad (6.6.7)$$

The practical experience shows that the values of the parameters α, β and γ are subject to construction conditions, pipe materials used and some other local conditions. In this study they are so determined :

$\alpha=9.72$, $\beta=1.635$ and $\gamma=2.05$.

Considering that α, β, γ and l_j $(j=1, \cdots\cdots, n)$ are all held constant, the above objective function can be reduced to the following function :

$$\text{Minimize } w=\sum_{j=1}^{n} \delta_j\ d_j^{\beta} \qquad (6.6.8)$$

where $\delta_j=\alpha l_j$.

6.6.4 Formulated Model

The above-formulated model is summarized.

<u>Objective Function</u>

$$\text{Minimize } w=\sum_{j=1}^{n} \delta_j\ d_j^{\beta} \qquad (6.6.8)$$

<u>Constraints</u>

$$\overline{A} f=\overline{q} \qquad (6.6.2)$$

$$A^t H=h \qquad (6.6.3)$$

$$H_i+L_i \geq \overline{H}_i \quad (i=1, \cdots\cdots, m_0) \qquad (6.6.4)$$

$$h_j=C d_j^{a} f_j^{b} l_j \ (j=1, \cdots\cdots, n) \qquad (6.6.5)$$

$$d_j \geq d_0 \ (j=1, \cdots\cdots, n) \qquad (6.6.6)$$

6.7 Solution Algorithms

6.7.1 Preliminary Discussion

The above-formulated model is a class of nonlinear programming. Before straightfoward applications of various techniques developed for solving different types of nonlinear programming problems, we shall examine the mathematical structure of the model with a view to develop some proper solution algorithms which will make best use of the specified structure of the model.

The conspicuous features of the model are pointed out.

(i) The reduced incidence matrix $\overline{A}$ can be partitioned such that

$$\overline{A}=[E \mid G] \qquad (6.7.1)$$

where E is the m_0-1 square matrix which contains a subset of all branches such that every node is connected but no loops are formed; this subset is called a spanning tree; and the associated cotree is represented by G.

Combining Equations (6.6.2) and (6.7.1), and partitioning f in accordance with Equation (6.7.1) into f_1 and f_2 , we get

$$E f_1+G f_2=\overline{q} \qquad (6.7.2)$$

It is apparent that the determinant $|E|$ of a tree is ± 1, since it is always possible to make E upper triangular with ± 1 elements on the main diagonal by giving any node the index number of its preceding branch, starting with no label at a reference node. Therefore E is nonsingular and its inverse E^{-1} exists. By multiplying Equation (6.7.2) by E^{-1}, we get

$$f_1 = E^{-1}\overline{q} - E^{-1} G f_2 \qquad (6.7.3)$$

(ii) The optimal directions of the pipe flows and in consequence those of the head losses can be a priori inferred by allowing for the topographic elevation contours, because we are assuming that no pumping stations are installed at the nodes other than the reference node (one of the supply nodes). Accordingly without loss in generality we can set

$$f_j \geqq 0 \quad (j=1, \cdots\cdots, n) \qquad (6.7.4\text{-}1)$$

provided that the signs of a_{ij} which are the components of the reduced incidence matrix $\overline{A}$ are predetermined so that they represent the optimal flow directions. The above equation can be rewritten as follows.

$$f_1 \geqq 0, \ f_2 \geqq 0 \qquad (6.7.4\text{-}2)$$

(iii) Substitution of Equation (6.7.3) into (6.7.4-2) gives

$$E^{-1}\overline{q} - E^{-1} G f_2 \geqq 0 \qquad (6.7.5\text{-}1)$$

$$f_2 \geqq 0 \qquad (6.7.6)$$

That is,

$$E^{-1} G f_2 \leqq E^{-1}\overline{q} \qquad (6.7.5\text{-}2)$$

$$f_2 \geqq 0 \qquad (6.7.6)$$

The above discussion tells us that the linear set of equality constraints of Equation (6.6.2) can be converted to a linear set of inequality constraints of Equations (6.7.5-2) and (6.7.6), thereby the concerned variables are reduced to $n-m_0+1$.

(iv) In likewise by substituting Equation (6.7.3) into (6.6.5), it can be rewritten as

$$h = g\,(d, f_2) \qquad (6.7.7)$$

where $d = (d_1, \cdots\cdots, d_n)$.

Combining Equation (6.6.3) and (6.7.7) we get the following set of nonlinear equations.

$$A^t H = g\,(d, f_2) \qquad (6.7.8)$$

(v) The remaining constraints of Equations (6.6.4) and (6.6.6) are sets of linear inequalities with respect to the explicit variables, H_i $(i=2, \cdots\cdots, m_0)$ and d_j $(j=1, \cdots\cdots, n)$, respectively.

(vi) Taking account of the above findings of (i) through (v), our solution mechanism should be so structured that it might efficiently deal with both the linear inequality constraints and the nonlinear ones. One of the sophisticated techniques for tackling with this kind of problem is, as has already be seen in Chapter 3, to combine two classes of nonlinear solution techniques, namely 1 one which works well for solving the problem of optimizing nonlinear objective function which are subject to a set of linear inequality constraints; and ② one which incorporates the nonlinear constraints into a modified objective function by the use of penalty factors.

In light of these considerations we have developed two types of solution algorithms based on two different ways of incorporating penalty factors. The detail explanation of these algorithms will be given in 6.7.2 and 6.7.3.

6.7.2 Solution Algorithm I

Solution Algorithm I deals with the following reduced nonlinear programming.

$$\text{Minimize } w=\sum_{j=1}^{n} \delta_j d_j^{\beta} \qquad (6.6.8)$$

subject to both the linear constrints :

$$\hat{G} f_2 \leq \hat{g} \qquad (6.7.5\text{-}3)$$

$$f_2 \geq 0 \qquad (6.7.6)$$

$$d_j \geq d_0 \ (j=1, \cdots\cdots, n) \qquad (6.6.6)$$

$$H_i + L_i \geq H_i \ (i=1, \cdots\cdots, m_0) \qquad (6.6.4)$$

and the nonlinear constraints :

$$M(d, f_2, H) = A^t H - g(d, f_2) = 0 \qquad (6.7.8\text{-}1)$$

where $\hat{G}=E^{-1}G$ and $\hat{g}=E^{-1}g$.

This algorithm combines the method of penalty factors developed by Fiacco and MacCormick[13)14)] (the internal point unconstrained minimization techniques) with the method of gradient projection due to Rosen[15)16)]. The former methods provides the main algorithm whereas the latter serves as the sub-algorithm.

1) Main Algorithm

Step 1

Find anyhow an arbitrary feasible solution $(d^{(1)}, f_2^{(1)}, h^{(1)}, H^{(1)})$ to the above nonlinear problem. This feasible solution can be easily found in the manner as follows. First for arbitrarily selected values for a set of variables, d_j $(j=1, \cdots\cdots, n)$ and f_{j_k} $(k=1, \cdots\cdots, n-m_0+1)$, check its feasibility. If first trial is found to be infeasible, update again the values for these variables and proceed in the same fashion until a feasible solution is obtained.

Step 2

For a properly predetermined penalty factor $r^{(K)}$ (K denoting the number of iterations of the process being discussed now), convert the above nonlinear programming problem into the problem of optimizing a nonlinear objective function with a linear set of constraints. That is,

$$\text{Minimize } F^{(K)}=w-\sum_{j=1}^{n} \frac{1}{r^{(K)}} \{ M_j (d, f_2, H) \}^2 \qquad (6.7.9)$$

subject to the linear constraints.

$$\hat{G} f_2 \leq \hat{g} \qquad (6.7.5\text{-}3)$$

$$f_2 \geq 0 \qquad (6.7.6)$$

$$d_j \geq d_0 \ (j=1, \cdots\cdots, n) \qquad (6.6.6)$$

$$H_i + L_i \geq \overline{H}_i \ (i=1, \cdots\cdots, m_0) \qquad (6.6.4)$$

where $M_j (d, f_2, H)$ $(j=1, \cdots\cdots, n)$ are the components of vector function defined by

Equation (6.7.8-1).

The above-formulated model will be handled by Sub-algorithm I-1 based on the method of Gradient Projection due to Rosen. This will be explained later.

By starting from the initial base point $(d^{(K)}, f^{(K)}, h^{(K)}, H^{(K)})$ which has been obtainted beforehand, solve the above nonlinear programming problem by the said sub-algorithm. Then we take the optimum of this problem $(\mathring{d}^{(K)}, \mathring{f}_2^{(K)}, \mathring{h}^{(K)}, \mathring{H}^{(K)})$ as our next initial base point $(d^{(K+1)}, f^{(K+1)}, h^{(K+1)}, H^{(K+1)})$.

At the first iteration $(K=1)$, we skip step 4 and proceed to step 5. Otherwise we go to step 4.

Step 4

Compare the old optimum of the penalty function $F^{(K)}$ with the new one of $F^{(K+1)}$ and check whether the following condition holds.

$$|F^{(K+1)} - F^{(K)}| < \eta \quad \cdots\cdots (6.7.10)$$

where η is a predetermined tolerance for the convergence of the algorithm.

If the above condition holds, our procedure terminates and we take this optimum as our desired one. Otherwise we step up to step 5.

Step 5

Reset the value of $r^{(K)}$ as

$$r^{(K+1)} = \frac{r^{(K)}}{T^{(K)}} \quad \cdots\cdots (6.7.11)$$

where $T^{(K)}$ is the modification factor of $r^{(K)}$ to give a series of $r^{(1)} > r^{(2)} > \cdots\cdots > r^{(K)} > r^{(K+1)} > \cdots\cdots$ which converge to zero.

Then we return to step 2 and the procedures should be repeated between steps 2 and 5 until the above convergence condition is found to be satisfied.

That is all for the outlined process of Solution Algorithm I. Additionally Sub-algorithm I-1 will be explained.

2) Sub-algorithm I-1

This algorithm utilizes the method of Rosen, which was developed to handle the following nonlinear programming problem.

$$\text{Minimize } z = f(\boldsymbol{x}) \quad \cdots\cdots (6.7.12)$$

subject to

$$A\boldsymbol{x} \geq \boldsymbol{b} \quad \cdots\cdots (6.7.13\text{-}1)$$

or

$$\sum_{j=1}^{n} a_{ij} x_j - b_i \geq 0 \quad (i=1, \cdots\cdots, m) \quad \cdots\cdots (6.7.13\text{-}2)$$

where $f(\boldsymbol{x})$ is a nonlinear objective function with respect to $\boldsymbol{x}(x_1, \cdots\cdots, x_n)$.

The Rosen's method uses certain rules of computation, starting from a base point $\boldsymbol{x}^{(1)}$, with r operative constraints, l of which $(l \leq r)$ are to be associated with $\boldsymbol{x}^{(1)}$. These rules are :

(i) The components of the gradient, $\frac{\partial z}{\partial x_j}$, are evaluated at the point $\boldsymbol{x}^{(1)}$.

(ii) Calculate the projection of the gradient on to the intersection of all the planes associated with the point $\boldsymbol{x}^{(1)}$. (Initially, let l be equal to r. In case l is zero, the gradient projection is the gradient itself.) This projection determines a direction in space which is constrained to lie along the l associated planes. This is the direction expressed as

$$t_i = \frac{-(z_{x_j} + \sum_{k=1}^{l} \lambda_{m_k} \frac{\partial g_{m_k}}{\partial x_j})}{\sum_{j=1}^{n} (z_{x_j} + \sum_{k=1}^{l} \lambda_{m_k} \frac{\partial g_{m_k}}{\partial x_j})^2} \quad \text{(6.7.14)}$$

with the λ_{m_k} determined by the l equations :

$$\sum_{j=1}^{n} \sum_{k=1}^{l} \lambda_{m_k} \frac{\partial g_{m_k}}{\partial x_j} \frac{\partial g_{m_k}}{\partial x_j} = \sum_{j=1}^{n} \frac{\partial g_{m_k}}{\partial x_j} z_{x_j} \quad \text{(6.7.15)}$$

where $g_{m_k} = \sum_{j=1}^{n} a_{m_{kj}} x_j - b_{m_k}$ (6.7.16)

(iii) If at least some of the t_i are different from zero, initiate a search in the direction of the gradient projection until a constraint boundary is encountered. Call this point $\mathring{x}^{(1)}$, a temporary values. ① If z ($\mathring{x}^{(1)}$) is an improvement when compared with z ($x^{(1)}$) the iteration is complete and $x^{(2)}$ is set equal to $\mathring{x}^{(1)}$. Step (i) is started again, with the newly encountered constraint included in the list of associated planes. ② If z ($\mathring{x}^{(1)}$) is not an improvement, find that point along the line connecting $x^{(1)}$ and $\mathring{x}^{(1)}$ which yields the best value of z. This is one dimensional search. Once $x^{(2)}$ is found, we return to step (ii).

(iv) If the projection along the l planes vanishes, there are no components of the gradient in the l planes. This implies that the gradient is perpendicular to all these planes. In other words, we have

$$\frac{\partial z}{\partial x_j} + \sum_{k=1}^{l} \lambda_{m_k} \frac{\partial g_{m_k}}{\partial x_j} = 0 \quad \text{(6.7.17)}$$

Two courses of action are required in this event. ① If all $\lambda_{m_k} \geqq 0$, the point $x^{(2)}$ satisfies all the Kuhn-Tucker conditions for a minimum. The point is then accepted as a local optimum. ② If the λ_{m_k} do not all have the same sign, define a new set of planes to be associated with $x^{(1)}$ by deleting from the set of l some single plane for which $\lambda_{m_k} < 0$ and return to step (ii).

6.7.3 Solution Algorithm II

This algorithm is a variant of Solution Algorithm I with the difference that it uses a different type of penalty factors developed by Powell. That is to say that the penalty function consists of two kinds of penalty factors, σ_j and θ_j ($j=1, \cdots\cdots, n$), assigned to each of the constraints to be incorporated into it.

$$\text{Minimize } \Phi^{(K)} = w - \sum_{j=1}^{n} \sigma_j^{(K)} \{M_j (d, f_2, H) + \theta_j^{(K)}\}^2 \quad \text{(6.7.18)}$$

subject to the linear constraints :

$$\hat{G} f_2 \leqq \hat{g} \quad \text{(6.7.5-3)}$$

$$f_2 \geqq 0 \quad \text{(6.7.6)}$$

$$d_j \geqq d_0 \ (j=1, \cdots\cdots, n) \quad \text{(6.6.6)}$$

$$H_i + L_i \geqq \overline{H}_i \ (i=1, \cdots\cdots, m_0) \quad \text{(6.6.4)}$$

The above-formulated model will be handled by the same sub-algorithm, i.e., Sub-algorithm II-1 of Solution Algorithm II.

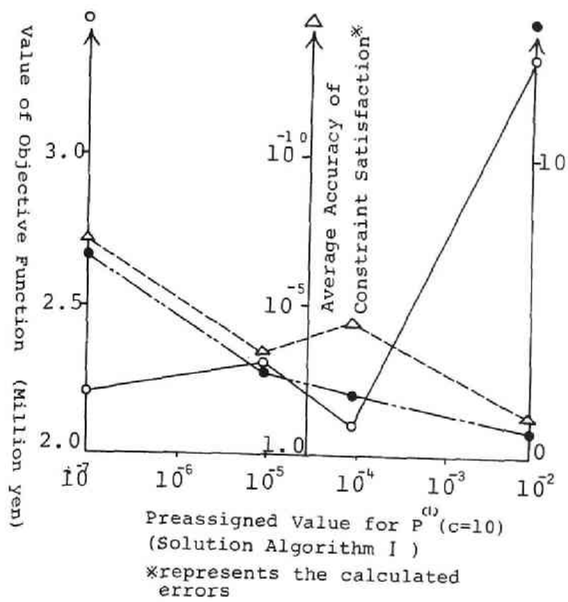


Fig. 6.8.2 Effect of Initial Penalty Value on Computational Efficiency (Solution Algorithm I)

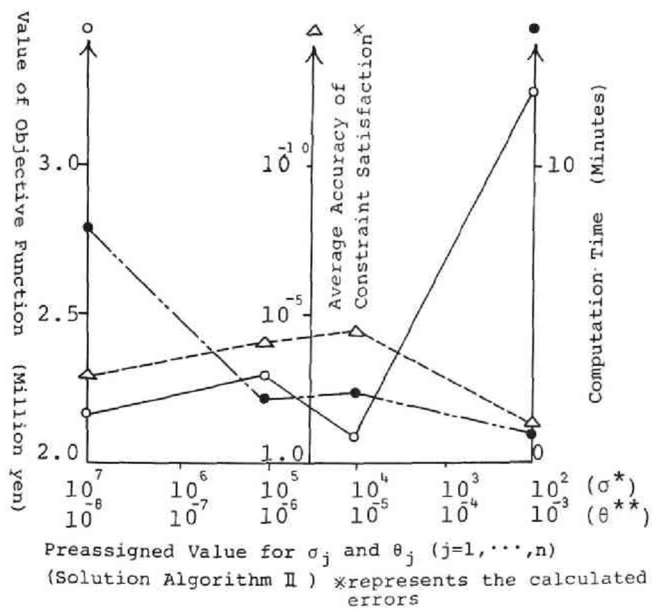


Fig. 6.8.3 Effect of Initial Penalty Values on the Computational Efficiency (Solution Algorithm II)

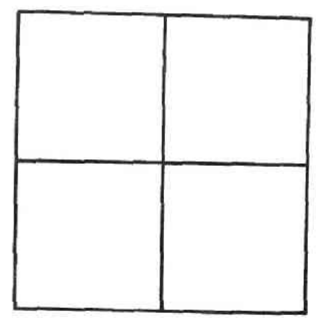


Fig. 6.8.1 Window-type Network

6.8 Model Application I (Comparative Study of Two Types of Solution Algorithms)

At this point of discussion we shall study the potentials and efficiency of the two kinds of solution algorithms developed by the authors. For brevity of analysis a simple nine-node network (including one supply node) with twelve pipes is considered as a sample network. We shall name the network of this configuration the "window-type network". (See Figure 6.8.1,)

The results of the computations derived from the same initial base point, while starting with different initial values set for the penalty factor(s), are listed in Figures 6.8.2 and 6.8.3. Close scrutiny of these figures leads to the following observations.

(i) Different searches starting with different values set for the penalty factors, where the same initial base point is selected, lead to different local minima. In a likewise manner if the searches start from different initial base points, where the preassigned initial value(s) for the penalty factor(s) are the same, it reaches different local minima. These different optima should be attributable to the nonconvexities involved in both the objective function and the constraints in the model. By comparison of these diffent optima and choice of the best one, it is found that the most adequate order of the initial value for the penalty factor is identified with 10^{-4} to 10^{-5} for Solution Algorithm I, and 10^4 to 10^5 ($\sigma_j^{(1)}$) and 10^{-5} to 10^{-6} ($\theta_j^{(1)}$) for Solution Algorithm II.

This is also the case with the calculated constraint satisfaction errors defined by $\frac{1}{n}\sum_{j=1}^{n} M_j(d, f_2, H)$, which would theoretically be equal to zero.

(ii) This also implies that the adequate order of $\sigma_j^{(1)}$ used in Solution Algorithm II is found to be roughly the reciprocal of that of $r^{(1)}$ used in Solution Algorithm II. This seems to be quite reasonable, because $r^{(K)}$ is theoretically identical with $\frac{1}{\sigma_j}(K)$, if the remaining factor, $\theta_j^{(K)}$, is neglected.

(iii) There is slight difference between the results of both algorithms, so far as the computation time and the calculated errors are concerned. But more rigorously, Solution Algorithm I seems to take a little shorter time than Solution Algorithm II.

(iv) In light of these considerations the subsequent two application studies will be approached by Solution Algorithm I.

6.9 Model Application 2 (In-depth Analysis of Window-type Network)

In this section our mathematical model will be applied to the optimal design problem of the "window-type" distribution network system. Our focus will be placed on the difference in the results derived from different design conditions with a view to provide some basic guidelines in which the optimal designs should be considered. With this understanding sensitivity analysis will be made by virtue of Solution Algorithm I for the reason as explained just before. The main reasons for the choice of the window-type network as a sample are that ① in spite of its simple configuration it can be considered the most typical and simple structure, while possessing the characteristics of more complicate ones. ② This or its variants find(s) itself (or themselves) in frequent uses in practical designs such as the design of the distribution system of a small-scale residential section.

The calculation results for different cases are shown in Figures 6.9.1 through 6.9.7. From them the following observations are made :

(i) Given the piezometric pressure head H_1 at the supply node 1, the calculated costs increase roughly in proportion to the increase in the inflow q_1. (See Figure 6.9.1.)

(ii) On the contrary given the inflow q_1, the cost decreases almost in inverse proportion to the increase in H_1. This provides a basis for the incorporation of the costs associated with the installation of pumping facilities or a distribution reservoir, because some artificial elevation of the pressure head at the supply node involves the installation of such facilities. (See Figure 6.9.2.)

(iii) Given the conditions of both the inflow and the supply head and the location of the supply node being held fixed, the central horizontal pipelines denoted by 6 and 7 as illustrated in Figure 6.9.3 are moved to three different locations. If the other input conditions are the same, the network denoted by B_1 leads to the most economical system, whereas C_1 the most uneconomical one. The standard network configuration, A_1, proves to be the medium.

The maximum-diameter pipe is found to be one of the central vertical axes, that is, pipe 4, for both Networks A_1 and B_1, and pipe 9 for Network C_1.

(iv) Given both a network configuration, and supply node conditions associated

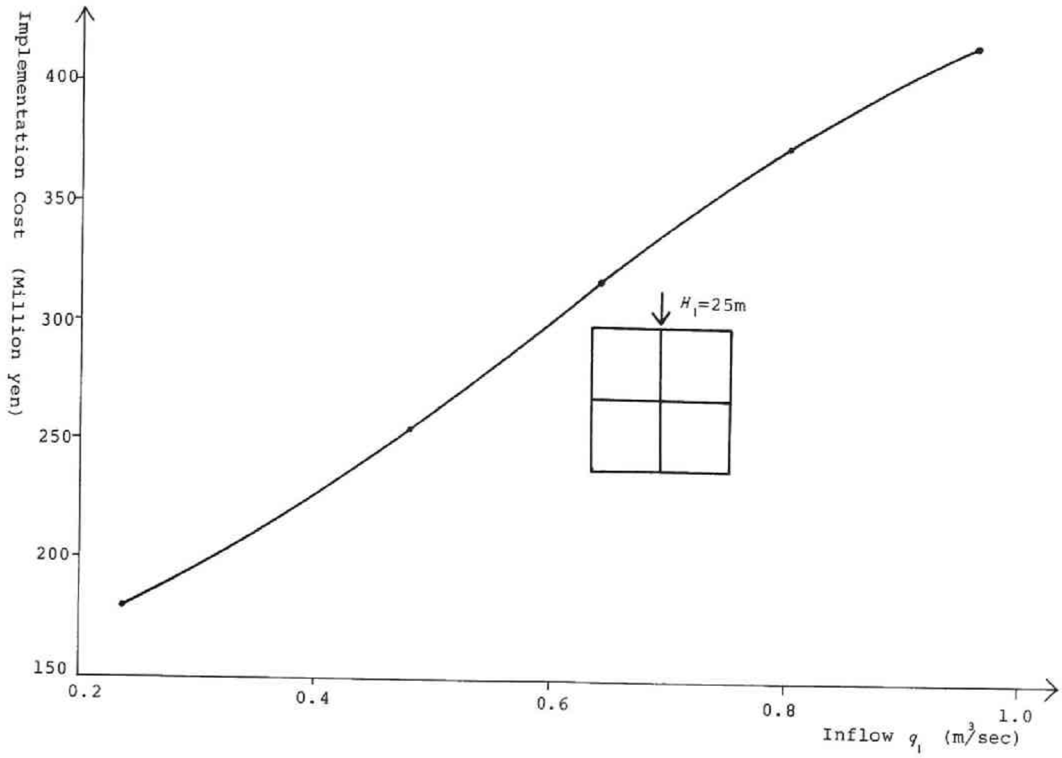


Fig. 6.9.1 Effect of Change in q_1 on Total Cost

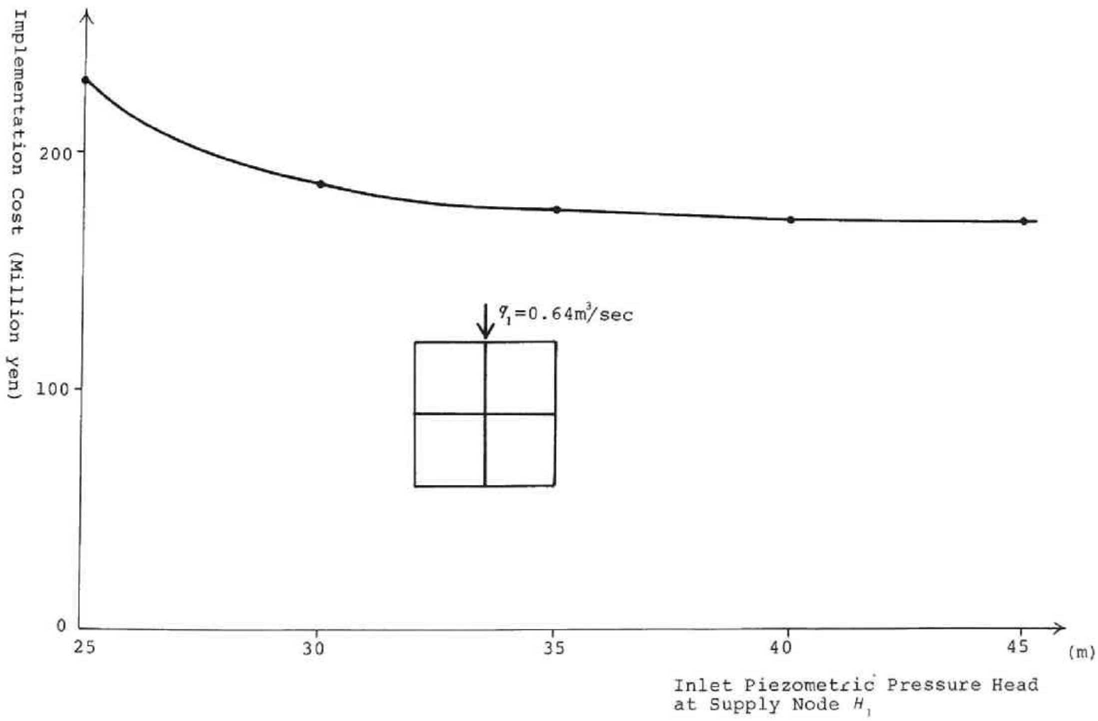


Fig. 6.9.2 Effect of Change in H_1 on Total Cost

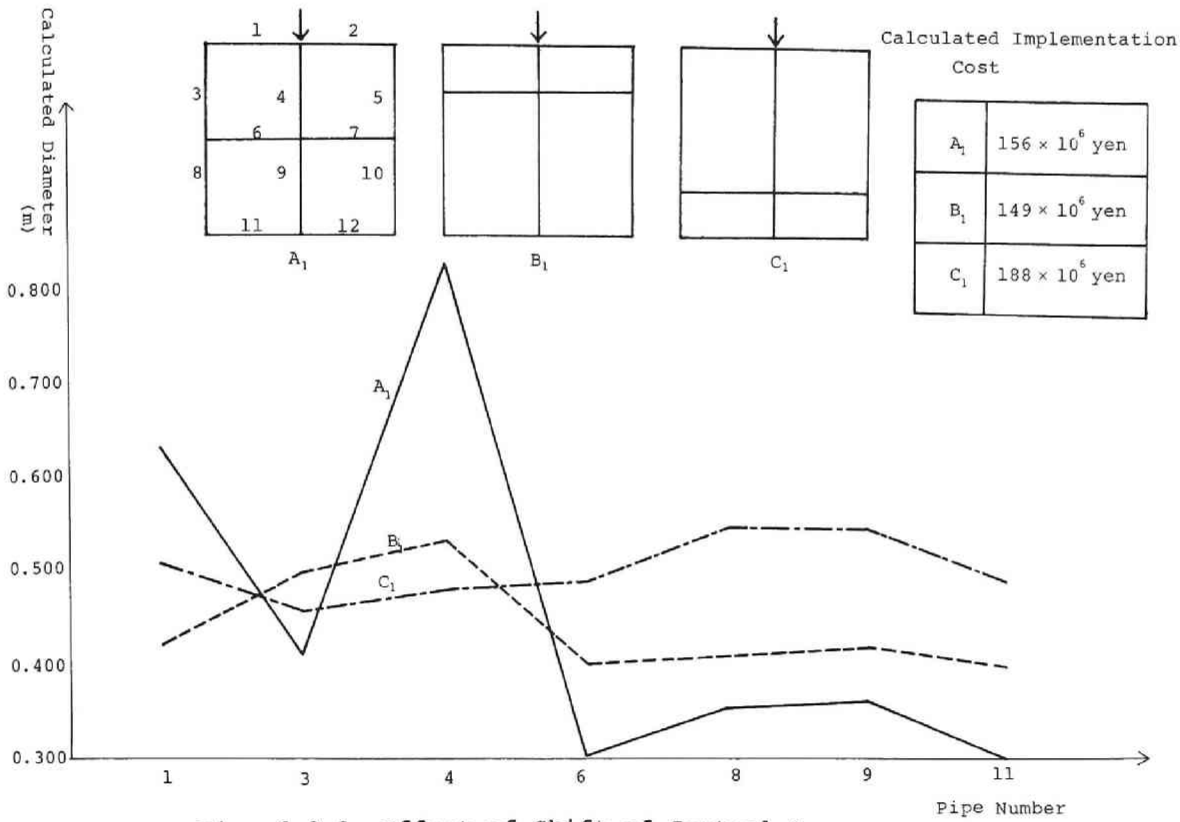
with the inflow and pressure head, the point of supply is changed. (See Figure 6.9.4.) The results are that ① choice of the node, upper central, yields more economical system than that of the upper left (or the upper right since the network is given symmetrical with respect to the central vertical pipelines). ② The maximum-diameter pipe is identified with pipe 4 for the choice of the upper-central point as a supply node, whereas pipes 1 and 3 for that of the upper left.

(v) Next let us consider those cases where the number of supply points are augmented, thereby the total quantity of supply as well as the other input conditions being fixed. Study of the results as shown in Figure 6.9.5 shows : ① As the supply points are multiplied, the total costs decrease. This seems natural, because it contributes to the improvement in the hydraulic conditions of the entire network. In this connection observation should be made that the pluralism of supply points involves increased investments in the additional facilities, such as those pipes jointing at the supply nodes, pumping facilities, distribution reservoirs,etc. Therefore if these costs are included into the total costs, we cannot easily say which alternative is preferred and in this sense further examinations are needed. ② The maximum-diameter pipe is found to be pipe 4 consistently for all the cases, although the multiple supply nodes tend to reduce the maximum diameter by about 40 percent and lead to the equalization of the entire pipe diameters.

(vi) In connection with the above cases let us compare the results for those cases where two supply points are given, one being fixed at the upper-left corner, the other moved to different locations. (See Figure 6.9.6.) Then the following would be clearly understood. ① If the discussion is confined to some ideal flat topography, the system becomes more inexpensive as the points of supply are further remote from each other. ② But it should also be noted that in practice the locations of supply points can seldom be determined without allowing for the locations of distributing reservoirs and other related facilities.

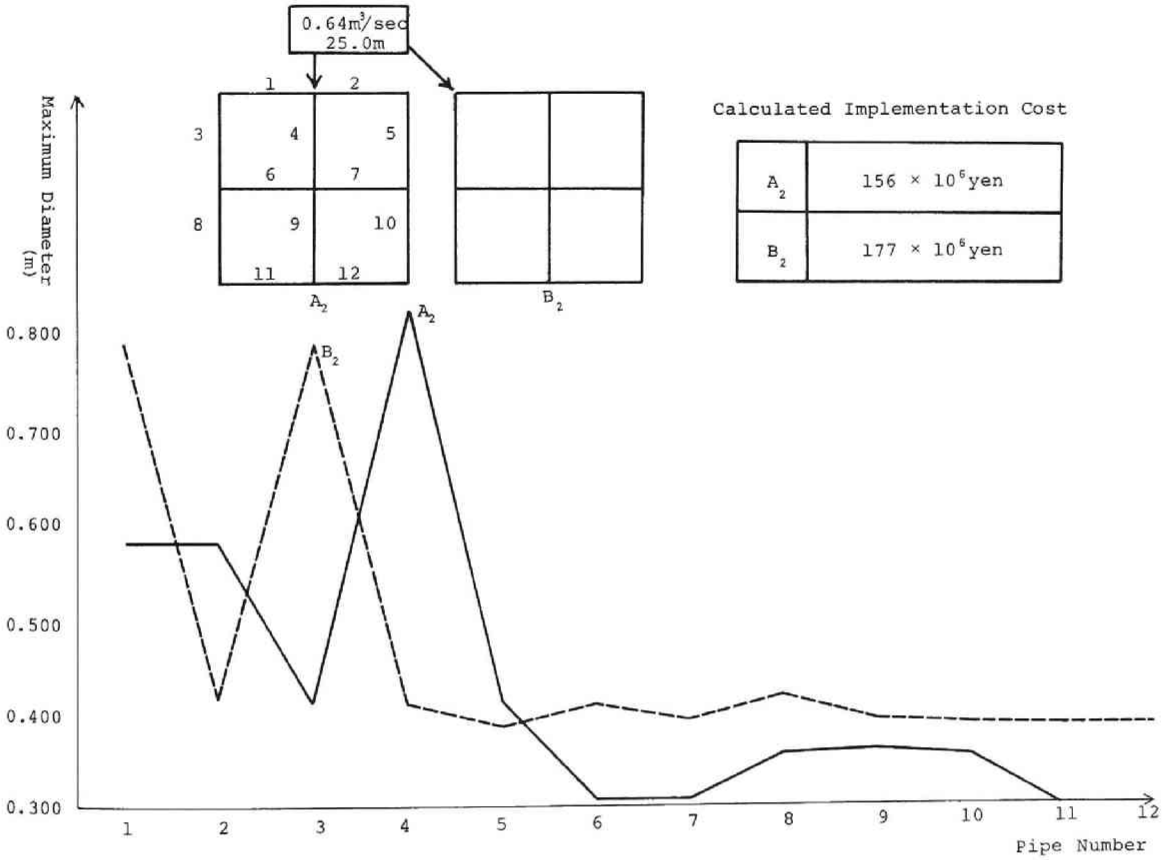
(vii) To supplement the above findings we shall consider a further complicate network denoted by B_5. (See Figure 6.9.7.) What comes to be clear from this are the following. ① The results derived from both the window-type network and the complex one have proven to be similar. ② One difference is that the latter leads to a more equalized-diameter network than the former. ③ It also differs from the former in it that it yields a more expensive network mainly because of some cross-type feeder distributors added to the former configuration.

(viii) Finally on the basis of this refined network, attention will be turned to that case where the topography is not flat and the difference in the elevation is given as shown in Figure 6.9.8. Close analysis of this reveals : ① If a moderate geographical slope is given from the point of supply node to the demand nodes, we can get a more economical distribution system. This seems reasonable, judged from the more favorable hydraulic conditions given by the topography. ② In spite of the loss in the topographical symmetry, the results show that the symmetrical structure remains still held. It is inferred that a larger degree of dissymmetry would lead to the loss in the symmetrical structure of the calculated hydraulic conditions.



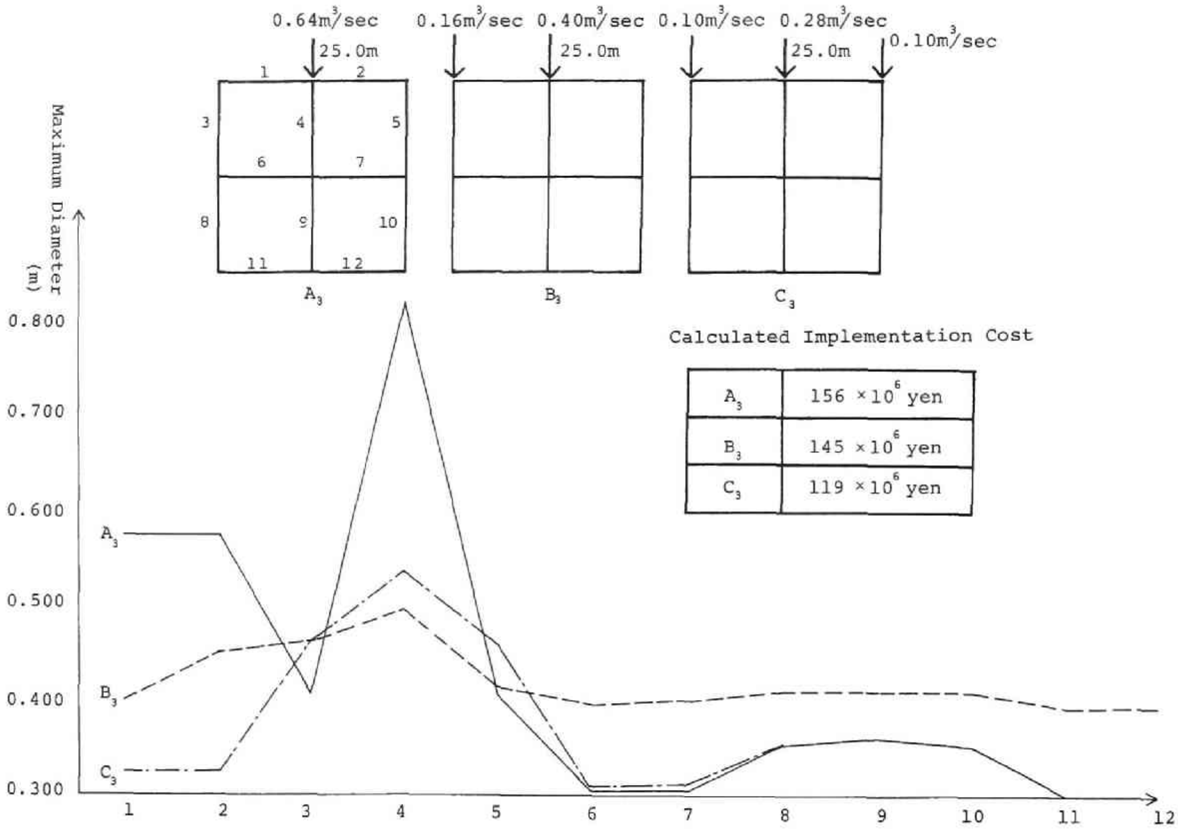
A_1	156×10^6 yen
B_1	149×10^6 yen
C_1	188×10^6 yen

Fig. 6.9.3 Effect of Shift of Central Axes on Total Cost and Hydraulic Conditions



A_2	156×10^6yen
B_2	177×10^6yen

Fig. 6.9.4 Effect of Shift of Supply Node on Total Cost and Hydraulic Conditions



Calculated Implementation Cost

A_3	156×10^6 yen
B_3	145×10^6 yen
C_3	119×10^6 yen

Fig. 6.9.5 Effect of Multiplication of Supply Nodes on Total Cost and Hydraulic Conditions

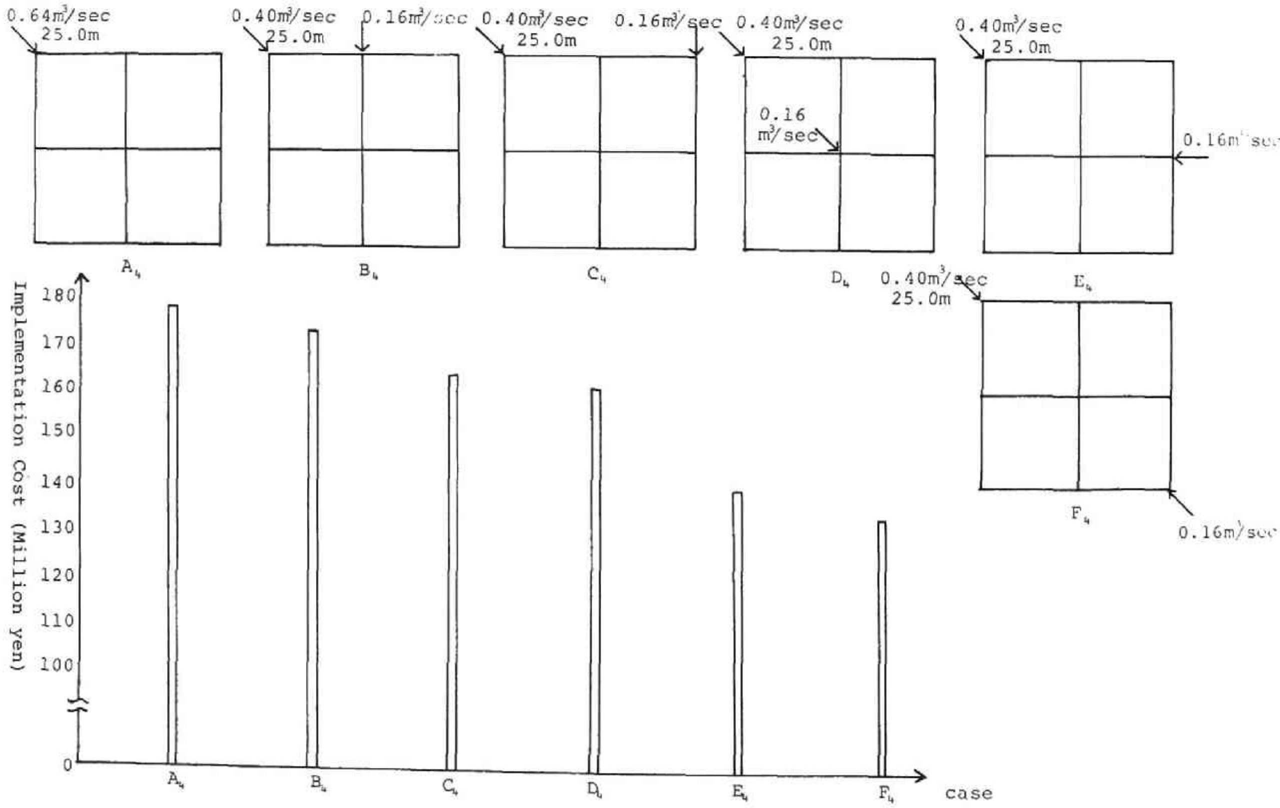
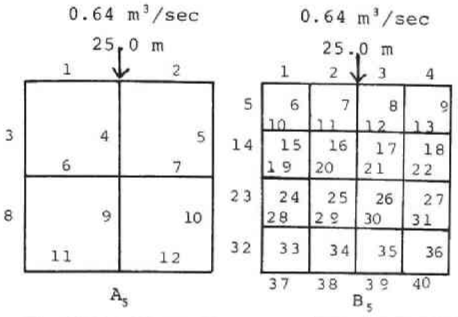


Fig. 6.9.6 Effect of Shift of Multiplied Supply Nodes on Total Cost



Calculated Implementation Cost

A_5	156×10^6 yen
B_5	188×10^6 yen

Calculated Diameter for Each Pipe (m)

A_5

1	0.575
2	0.575
3	0.407
4	0.822
5	0.407
6	0.302
7	0.302
8	0.354
9	0.362
10	0.354
11	0.300
12	0.300

B_5

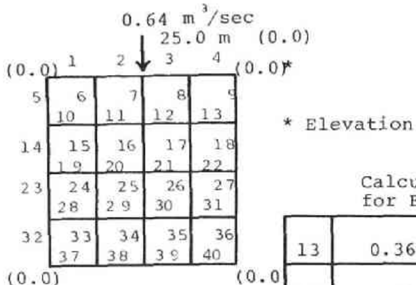
		13	0.367	27	0.300
		14	0.329	28	0.300
1	0.414	15	0.367	29	0.329
2	0.462	16	0.414	30	0.329
3	0.462	17	0.367	31	0.300
4	0.414	18	0.329	32	0.300
5	0.367	19	0.329	33	0.300
6	0.414	20	0.367	34	0.329
7	0.462	21	0.367	35	0.300
8	0.414	22	0.329	36	0.300
9	0.367	23	0.300	37	0.300
10	0.367	24	0.329	38	0.300
11	0.414	25	0.367	39	0.300
12	0.414	26	0.329	40	0.300

Fig. 6.9.7 Effect of Increase in Complexity of Network Configuration

6.10 Model Application 3 (Case Study on the Integrated Development of Area-wide Water Distribution System for Kakogawa and Takasago Cities)

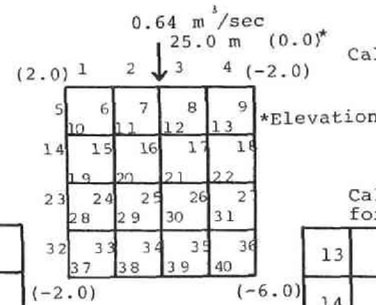
6.10.1 Preliminary Discussion

The remainder of this study will be devoted to the application of the model to another feature of the problem, that is, a kind of problem which will be encountered in the phase of planning an urban water supply system. Since our main theme discussed thoughout this thesis concerns area-wide development of water utilization facilities, whether at a local, regional or national level, the problem considered is limited to the integrated development of cross-boundary water distribution system, thereby the study area being selected as the local area covering parts of Cities of Kakogawa and Takasago as illustrated in Figure 6.10.1. This area consisting of two local districts called by the namesake of "Yoneda-cho", while one belonging to Kakogawa City and the other to Takasago City, reflecting the historical background such that they had constituted the same local administrative body before they were divided into two municipalities. In consequence this historical career is partially reflected in the existing water distributuion system which is presently used jointly by the two sections.



Calculated Diameter for Each Pipe (m)

1	0.414	13	0.367	27	0.300
2	0.462	14	0.329	28	0.300
3	0.462	15	0.367	29	0.329
4	0.414	16	0.414	30	0.329
5	0.407	17	0.367	31	0.300
6	0.414	18	0.329	32	0.300
7	0.462	19	0.329	33	0.300
8	0.414	20	0.367	34	0.329
9	0.367	21	0.367	35	0.300
10	0.367	22	0.329	36	0.300
11	0.414	23	0.300	37	0.300
12	0.414	24	0.329	38	0.300
		25	0.367	39	0.300
		26	0.329	40	0.300



Calculated Implementation Cost

A	188×10^6 yen
B	178×10^6 yen

Calculated Diameter for Each Pipe (m)

1	0.396	13	0.350	27	0.300
2	0.443	14	0.318	28	0.300
3	0.443	15	0.350	29	0.318
4	0.396	16	0.396	30	0.318
5	0.350	17	0.350	31	0.300
6	0.396	18	0.318	32	0.300
7	0.443	19	0.318	33	0.300
8	0.396	20	0.350	34	0.300
9	0.350	21	0.350	35	0.300
10	0.350	22	0.318	36	0.300
11	0.396	23	0.300	37	0.300
12	0.396	24	0.318	38	0.300
		25	0.350	39	0.300
		26	0.318	40	0.300

Fig. 6.9.8 Effect of Change in Geographic Topology

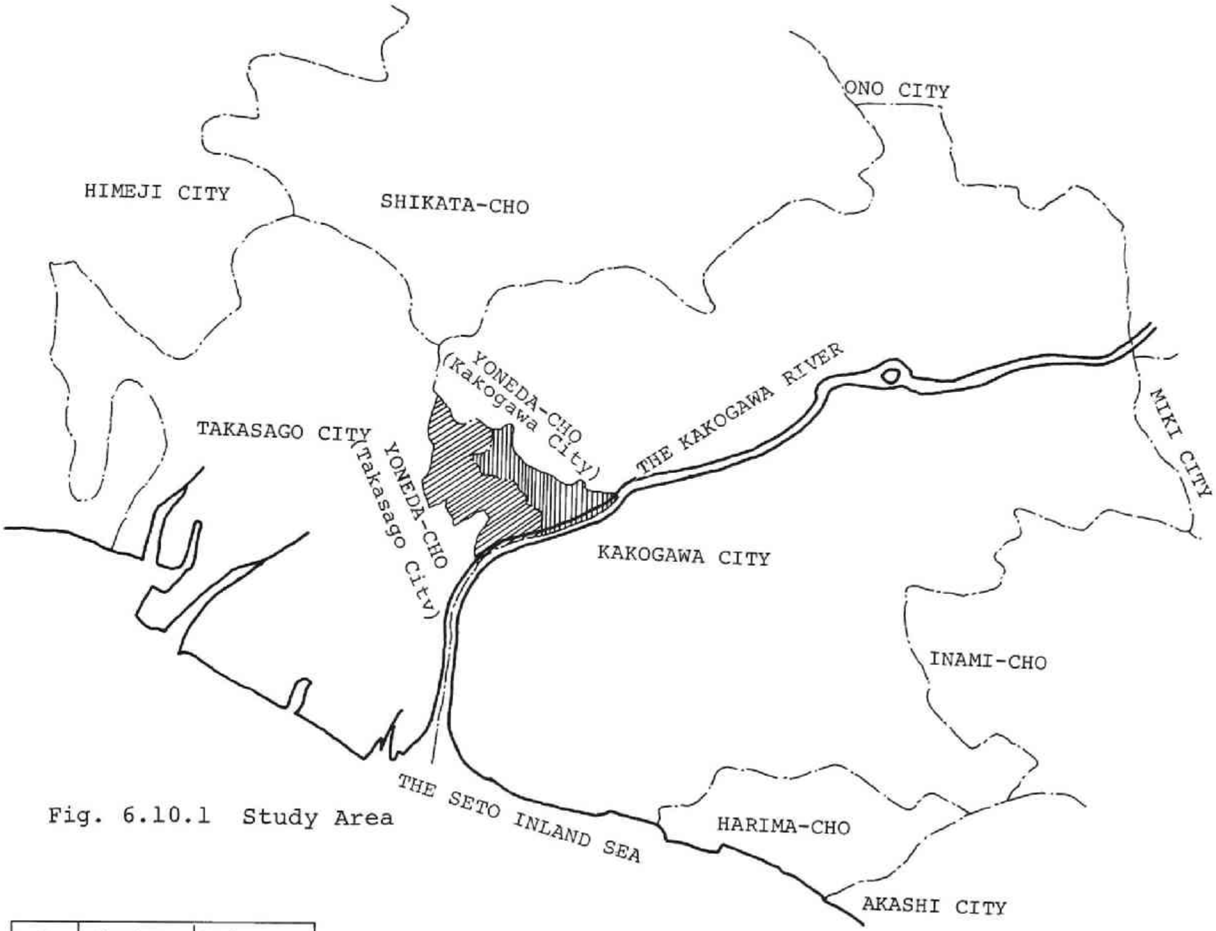


Fig. 6.10.1 Study Area

Node NO.	Design Outlet Flow	Node Elevation
	(10^{-2} m^3/sec)	(m)
C1	0.45	3.7
C2	3.22	3.5
C3	1.55	3.9
C4	1.09	6.0
C5	2.18	5.8
C6	1.72	4.7
C7	1.05	4.0
C8	1.21	4.5
C9	0.97	3.7
C10	1.79	3.5
C11	3.39	3.1
C12	1.58	3.5
C13	1.18	3.5
C14	0.82	3.8
C15	1.03	3.9
C16	1.50	3.9
Total	24.78	–

Node No.	Design Outlet Flow	Node Elevation
	(10^{-2} m^3/sec)	(m)
T1	0.45	3.7
T2	3.22	3.5
T3	0.41	3.9
T4	0.32	4.0
T5	1.43	3.5
T6	0.21	3.7
T7	1.58	3.5
T8	0.89	3.5
T9	0.42	3.8
T10	3.39	3.1
T11	1.50	3.9
T12	1.03	3.9
Total	14.86	–

Node No.	Design Outlet Flow	Node Elevation
	(10^{-2} m^3/sec)	(m)
K1	1.09	6.0
K2	1.14	3.9
K3	2.18	5.8
K4	0.73	4.0
K5	1.72	4.7
K6	0.37	3.5
K7	0.76	3.7
K8	1.21	4.5
K9	0.29	3.5
K10	0.45	3.8
Total	9.93	–

Table 6.10.1 Input Data (1) (Case C)
Table 6.10.2 Input Data (1) (Case T)
Table 6.10.3 Input Data (1) (Case K)

Pipe-line NO.	Pipeline Length
	(m)
C1	680
C2	1,200
C3	750
C4	400
C5	1,060
C6	400
C7	600
C8	430
C9	610
C10	990
C11	780
C12	1,300
C13	1,090
C14	90
C15	1,010
C16	510
C17	1,350
C18	230
C19	240
C20	440
C21	580
C22	1,080
C23	990

Table 6.10.4 Input Data (2) (Case C)

Pipe-line No.	Pipeline Length
	(m)
T1	680
T2	1,200
T3	780
T4	990
T5	400
T6	610
T7	90
T8	1,300
T9	1,090
T10	1,010
T11	1,030
T12	440
T13	240
T14	230
T15	580
T16	1,080
T17	990

Table 6.10.5 Input Data (2) (Case T)

Pipe-line No.	Pipeline Length
	(m)
K1	400
K2	750
K3	400
K4	1,060
K5	600
K6	610
K7	430
K8	990
K9	90
K10	510
K11	330
K12	1,010
K13	1,350
K14	230

Table 6.10.6 Input Data (2) (Case K)

Cost \ Case	Case K	Case T	Case C
Calculated Implementation Cost for Each Case	143.7	198.3	263.4
Total Cost	342.0		263.4

(Million yen)

Table 6.10.7 Calculated Implementation Costs

6. 10. 2 Identification of the Problem

In light of these considerations this jointed area was selected as the case study area, although we conceptualize a new development planning problem provided that the study area is presently not equipped with a distribution system and it will be implemented to meet the projected water demands as given in Tables 6.10.1 to 6.10.3. Other input values are also given in Tables 6.10.1 to 6.10.6.

Since our primary concern is to analyze the advantage and disadvantage of the integrated development system by comparison with the independent development system isolated from each other, we shall consider the following three cases, namely Case K where the isolated development of the distribution system for the section of Kakogawa City is considered, Case T where it is implemented for that of Takasago City, and lastly Case C where the joint implementation system is considered.

6.10.3 Calculation Results

Computations are conducted on the model by the use of Solution Algorithm I as used in the former study. Thereby amongst several trials with different initial values for the penalty factors, the best solution was singled out as our optimum.

The results for the three cases are shown in Figures 6.10.2 to 6.10.4 and Table 6.10.7. Comparative scrutiny of these figures leads to the following observations.

(i) So far as the total costs are concerned, the joint implementation system have proven to be more economical than the isolated implementation systems. The reduced total costs of the former system are found to be about 80 million yen, about 20 percent lower than the latter. This seems to be derived from the scale advantages of the former system.

(ii) The hydraulic conditions are also seen to be more favorable in the former system, because, for instance, the gap in the piezometric pressure heads is smaller for the former than for the latter, implying a higher attainment of equalization in hydraulic conditions at the extremities of the supply nodes.

(iii) At first blush one exception to this appears to be the diameters of pipes. That is to say that the gap between the maximal and minimal diameters tend

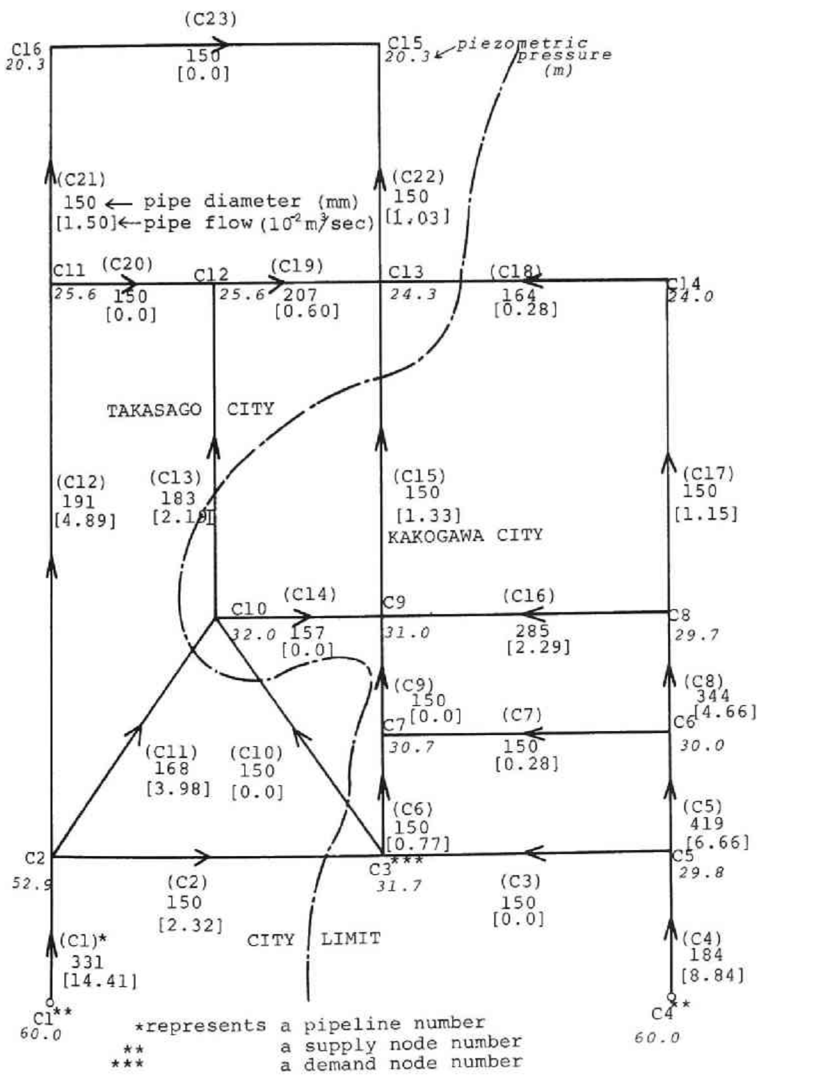


Fig. 6.10.2 Illustrated Results for Case C

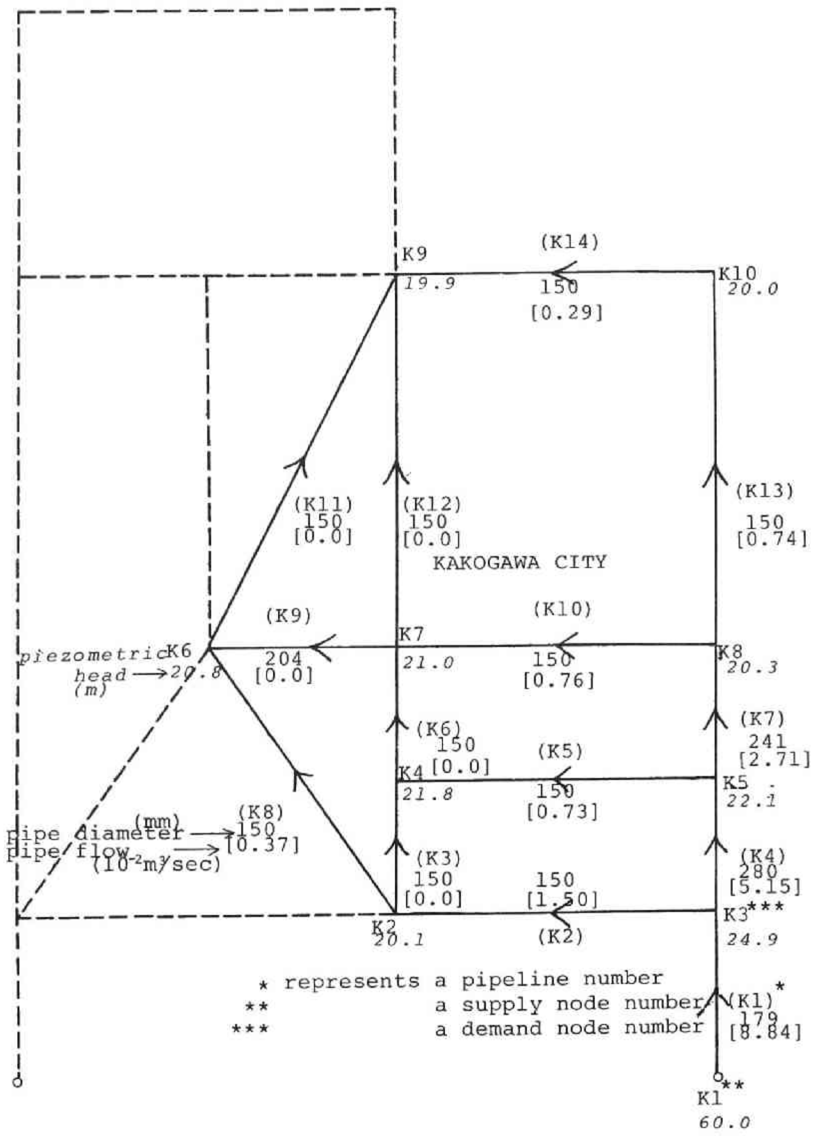


Fig. 6.10.4 Illustrated Results for Case K

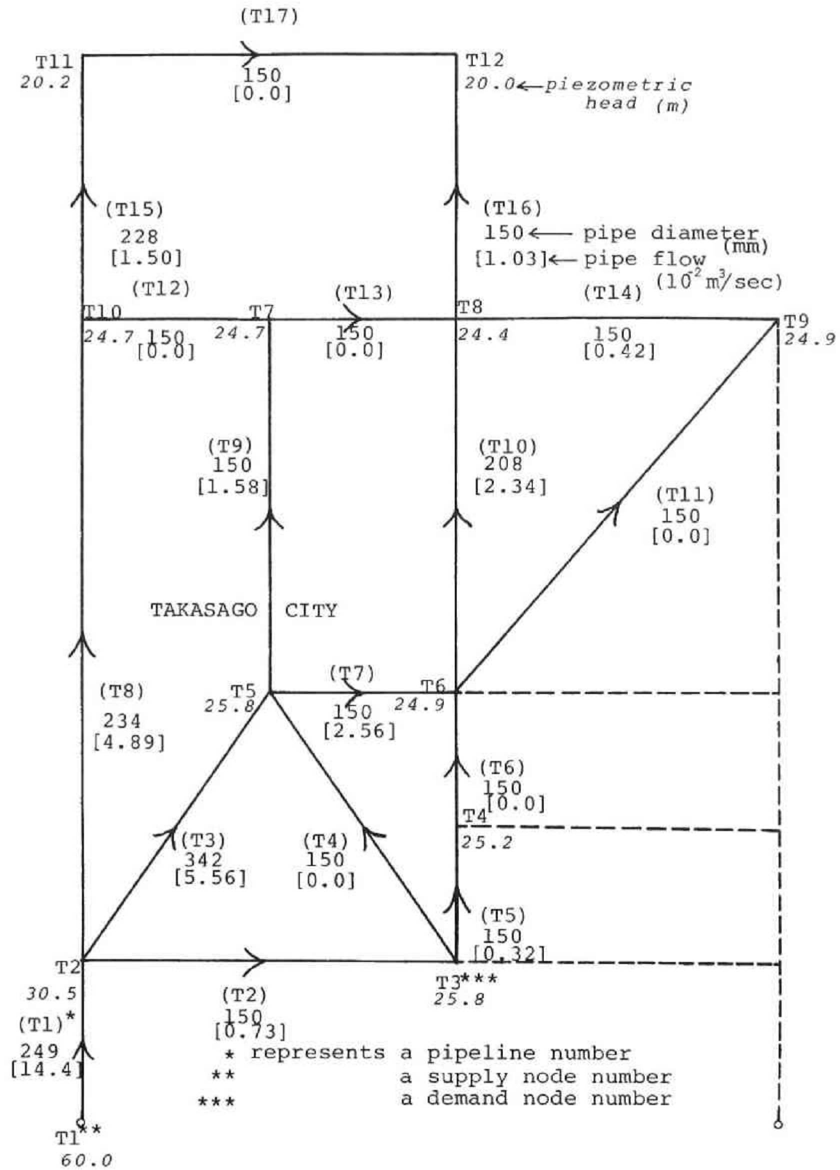


Fig. 6.10.3 Illustrated Results for Case T

to extend if the separate networks are combined. But we should also observe that another tendency typical of the joint implementation system is that there are several pipes of medium sizes (say, 250 to 300 mm) incorporated into the system, whereas the isolated implementation system is likely to be polarized to two extremes, i.e., the mazimum- and minimum-diameter pipes. Therefore the incorporated pipes of medium sizes which are considered to serve as supplementary mains seem to contribute to the equalization in the hydraulic conditions.

(iv) To sum up the above findings, the integrated water distribution system is found to be more advantageous, judged from the viewpoints of economy and hydraulic conditions.

(v) If other features of the problem, such as physical, administrative and managerial questions involved in the integrated development of an area-wide distribution system are also examined, our mathematical model would provide well-informed guidelines along which to consider this kind of problem in the planning phase, not to say in the design one.

6.11 Conclusion

To begin with, the important findings are summarized.

(i) The mathematical model presented in this study provided some basic frameworks for both the design and the planning of the water distribution system, if some effective solution algorithm could be developed.

(ii) In this viewpoint our first attention was offered to the development of those tools. For this purpose two different algorithms were developed by the authors with a view to obtain any adequate tools that would take a good advantage of the mathematical idiosyncrasy of the model.

(iii) Comparative study of the computation results stemmed from the applications of the different algorithms to a simple network, reveals that ①although there can be found no conspicuous differences in the adequacy of the techniques, we might well say that at the risk of a slight loss in the constraint satisfaction accuracy, Solution Algorithm I is preferred, because it takes a little shorter computation time than the other. ②In this connection such a practical consideration should be added that in practice available diameters for pipes are not continuous but discrete. Therefore if the rounded-off solution is found to exceed the range of errors, it makes no sense to stick to the more accurate solutions. Therefore it might be concluded that a priority should be given to Algorithm I. The author acknowledges that these points should receive further consideration.

(iii) On the basis of the above analysis our next concern went to the comparative study on the optimal design of the "window type" and its variants. Thereby our intention lied in it that it might offer an informative analysis, from which some fundamental guidelines for the optimal design could be derived.

(iv) As a result some important guidelines were obtained concerning ①the adequate locations of both arteries and complementary distributors, ②the effect of the change in the location(s) of the supply point(s) on the computation results, ③the effect of the increased complexity in the network configuration on the results, ④the effect of the difference in the geographical topology on the results, ⑤and other few points.

(v) At the last stage of our study we addressed ourselves to the area-wide development planning problem of the urban water distribution system. Case study was conducted on the local district covering parts of both Kakogawa and Takasago Cities in Hyogo Prefecture.

(vi) Close scrutiny of the results have led to the conclusions that ① the integrated water distribution system is found to be more advantageous, judged from the viewpoints of economy and hydraulic conditions. ② If further analyses will be made on the other features of the problem such as other technological examinations, administrative and managerial questions, etc., we could be better informed of this kind of planning problem.

(vii) Although the criterion for the optimality was established of the minimization of the total costs, the model with slight modification would be well adapted to other kinds of criteria such as maximization of the attainment of the equalization in the entire piezometric pressure heads. The author believes that the presented model finds itself in other various uses though untouched in this study. These include the capacity expansion problem of distribution network systems.[2]

(viii) Comprehensive analysis of the above findings have led the author to the conclusion that the model or its variants would serve as an effective tool for providing basic guidelines on both the design and the planning of the water distribution system.

References

1) Okada, N., and Yoshikawa, K.: Mathematical Modelling and Analysis of Water Distribution Network System, Proc. of J.S.C.E. (To Be Issued).

2) Okada, N., and Haruna, M.: A Mathematical Model for Capacity Expansion Problem of Inter-regional Water Supply Facilities, Memoirs of the Faculty of Engineering, Kyoto University, Vol.XXXVI, Part 3, July, 1974.

3) Sakurai, H.: Study on Modelling Water Distribution System, Graduation Thesis Presented to the Faculty of Engineering, Kyoto University, March, 1974 (In Japanese).

4) Nagano, M.: Systems Analysis of Simple Network for Water Distribution, Graduation Thesis Presented to the Faculty of Engineering, Kyoto University, March, 1975 (In Japanese).

5) Iwasaki, Y.: Systems Analysis of Capacity Expansion of Water Distribution Network System, Graduation Thesis Presented to the Faculty of Engineering, Kyoto University, March, 1976 (In Japanese).

6) Sueishi, T.: Theoretical Overview of Water Distribution Network Systems; Application of Gradient Method, Journal of Japan Society for Water Supply Works Engineers, pp.2-13, Apr., 1966 (In Japanese).

7) Jacobi, L.S.: Design of Optimal Hydraulic Networks, Journal of the Hydraulics Division, ASCE, Vol.94, No.HY3, Proc. Paper 5930, pp.647, May, 1968.

8) Nakajima, S.: Design of Water Supply Network Systems Based on the Shortest Route Principle, Proc. of JSCE, Vol.191, July, 1971 (In Japanese).

9) Watanatada, T.: Least-Cost Design of Water Distribution Systems, Journal of the Hydraulics Division, ASCE, Vol.99, HY2, Proc. Paper 9974, Sept., 1973.

10) Lam, C.F.: Discrete Gradient Optimization of Water Systems, Journal of the Hydraulics Division, ASCE. Vol.99, HY2, Proc. Paper 9804, June, 1973.

11) Haarhoff, P.C., and Buys, J.D.: A New Method for the Optimization of a Nonlinear Function Subject to Nonlinear Constraints, The Computer Journal, London, England, Vol.13, No.2, pp.178-184, May, 1970.

12) Fiacco, A.V., and McCormick, G.P.: The Sequential Unconstrained Minimization Technique for Non-linear Programming; A Primal-dual Method, Management Science, Vol.10, pp.360-366, 1964.

13) Fiacco, A.V., and McCormick, G.P.: Computational Algorithm for Sequential Unconstrained Minimization Technique for Non-linear Programming, Management Science, Vol. 10, pp.601-617, 1964.

14) Fiacco, A.V., and McCormick, G.P.: Extension of Sequential Unconstrained Techniques for Non-linear Programming, Equality Constraints and Extrapolation, Management Science, Vol.12, pp.816-818, 1966.

15) Rosen, J.B.: The Gradient Projection Method for Non-linear Programming, Part I, Linear Constraints, J. Soc. Ind. Appl. Math., Vol.8, pp.181-217, 1960.

16) Rosen, J.B.: The Gradient Projection Method for Non-linear Programming, Part II, Non-linear Constraints, J. Soc. Ind. Appl. Math., Vol.9, pp.514-532, 1961.

Chapter 7 Systems Analysis of Operational Control of Water Supply and Use System in Drought-time

7. 1 Introduction

In spite of increased recognition that water resources are limited and should be developed and used more effectively than they used to be, there seems to be many of the problems which have scarcely received proper attention of researchers and practitioners. One typical example is the problem of water droughts. The purpose of the studies included in this chapter is to present the development of a systematic methodology for determining the optimal policy for water supply and demand control problems. The term "optimal" in this context means a rule that leads to the operational policy most acceptable to the water users.

7. 2 Identification of the Problem

Since water supply cut-off is a degradation of water supply service, utmost efforts should be made to avoid it. But when it is found unavoidable by any means after a long continuation of water droughts, next best attempt should be made to alleviate as much as possible the extent of undesirable effects of water supply cut-off upon the water users.

In this chapter we shall take the position that in coping with water drought problems, operational control policies for alleviating unacceptable damages of droughts to the water users are as much important as long-range strategies for water resources development. In light of these considerations our attention will be exclusively placed on the explorement of systems approaches for finding the optimal rule for drought-time water supply and demand controls.

7. 3 Scope of the Research

This chapter contains three major studies. 7.4 presents a pilot approach to the analysis of drought-time water demand and use characteristics with specific reference to the behavioral characteristics of the water users[2]. An attempt is made to explore a methodology for measuring "acceptability" in terms of statistical variates. On the basis of this analysis follows presentation of a mathematical model in 7.5. Applicability and potentiality for further developments are also explained with a case study on Kyoto City[1][2]. 7.6 deals with a mechanism of the dynamic interactions between the water supply and water demand sectors during the periods of water droughts, thereby presenting a system dynamics approach to the problem with a case study on Toyonaka City in Osaka Prefecture[1][3].

The paper closes with our assessment of the usefulness of the fundamental methodology proposed here for analyzing the drought-time operational control problems.

7. 4 Statistical Analysis of Behavioral Characteristics of Water Users in Drought-time

In this section a statistical approach is presented for analyzing the behavioral characteristics of the water users in time of droughts. We shall construct a statistical model which will be applied to the analysis of the behavioral characteristics of the water users in Tenri City in Nara Prefecture. We owe most of our basic data to the survey conducted there by the Kinki Branch of the Ministry of Construction with a view to gathering basic data relevant to the evaluation of water supply and demand control operations[4]. In this survey attempt was made to carry out a questionnaire which was so designed that it could present a basic framework in which statiscal analyses would be made by use of the Multidimensional Quantification Analysis, developed by C.Hayashi[5][6]. In spite of much reliance on these data, or the same statistical technique used in that survey, the study that follows is unique and original in that it evolves an analytical approach from another angle, thus leading to different outputs and results. Another purpose of the study is to present a basis on which different systems approach will follow in 7.5 and 7.6.

7.4.1 Questionnaire

The questionnaire was so designed that it could collect basic information related to the behavioral characteristics of the water users in Tenri City, who had experienced right ahead of the time of the survey, one of the longest and severest droughts they ever had. The contents of the questionnaire are mainly related to the following problems:

(i) to identify the recognized service level of cut-off operations,

(ii) to comprehend the recognized service level of the provisional measures taken by the authority such as itinerant water-supply wagons, "save water" campaigns, etc,

(iii) to gauge the impact of fall in water supply on their everyday life,

(iv) to investigate the manner the water users adjusted themselves to the water-supply cut-offs.

Let us now proceed to the analysis of the results of the questionnaire.

7.4.2 Preliminary Discussion

1) On Cut-off Operation

We here define the cut-off ratio as the ratio of the cut-off amount to the normal water supply. The term "normal" in this context means the phase when water droughts never occur and there is no need to cut-off water supply. Then by way of illustration, Table 7.4.1 shows that for the same cut-off ratio, the water users are likely to prefer an operation by lowering pressures on supply, rather than "direct" cut-off operation for limited hours of the day. This seems to imply that the water users are apt to accept rather an unintermittent supply of water which leads to an indirect cut-off of water supply, whereas they are unwilling to have their water supply operated in an on-and-off way to cut-off supply.

Since the average of the recognized cut-off ratio was found to be 0.35 and that of the actual ratio 0.20, there tends to be a certain discrepancy between the actual cut-off ratio and the recognized one. This gap tends to become smaller with rise in the actual cut-off ratio. Interestingly, there seems to be a smaller gap found between the changing rate of the actual cut-off ratio from one day to another and that of the recognized one. (See Table

Operational Measures		Recognized Severeness: very severe	severe	medium	moderate	very moderate	Total
Number of Those Who Think the Operation Was :	Ordinary Operation	3 (1.7)	24 (13.4)	25 (14.0)	59 (33.0)	68 (37.9)	170 (100.0)
	Lowered Pressure	24 (8.0)	75 (25.0)	62 (20.7)	114 (38.0)	25 (8.3)	300 (100.0)
	Cut-offs for Limited Hours	23 (6.1)	137 (36.4)	87 (23.1)	110 (29.3)	19 (5.1)	376 (100.0)
	Both of the Two	64 (21.4)	137 (45.8)	44 (14.7)	44 (14.7)	10 (3.4)	299 (100.0)
	Complete Shutdown	6 (37.5)	7 (43.8)	1 (6.3)	2 (12.4)	0 (0.0)	16 (100.0)

Table 7.4.1 Recognized Severeness for Different Operational Measures

OPERATION PATTERN			STAGE: INITIAL	MIDDLE	FINAL
Actual Pattern of Cut-off Ratio			Severe 0.4~0.5	Medium 0.3~0.4	Moderate 0.1~0.2
Recognized Pattern	percentage of those who selected the above pattern	% 47.2	Severe	Medium	Moderate
		20.9	Medium	Severe	Medium
		7.5	Moderate	Medium	Severe
		24.4	Severe	Severe	Severe

Table 7.4.2 Comparison between Actual Operation Pattern and Recognized Pattern

7.4.2.) This means that the water users more easily recognize the present degree of cut-off, namely the cut-off ratio by recalling that of the preceding day by comparison. This provides a good theoretical basis for the following discussions where it will be assumed that the recognized condition of cut-off operations corresponds to its actual condition insofar as the qualitative characteristics of the operational conditions are concerned.

2) Acceptable Cut-off Ratio

Figure 7.4.1 which displays the upper limit to the acceptable cut-off ratio for different duration times of cut-off operations shows that:

(i) The average of the upper limit ratio decreases as the duration time extends longer.

(ii) The distribution patterns of the upper limit ratio for the duration time of 1 week and for that of 1 month are mutually similar, though the pattern for the duration time of 3 months completely differs from the above two.

(iii) This can be otherwise interpreted in two different manners. (1) The extent to which the water users manage to adjust themselves to cut-off operations, decreases as the duration time expands. (2) The water users become less willing to spare water as the duration time expands.

The reader is required to recall the above findings in the model-building discussions in 7.5 and 7.6.

7.4.3 Drought-time Behavioral Pattering by Multi-dimensional Quantification Method

Next we shall patternize the behavioral characteristics of the water users from a mathematico-statistical point of view. An approach is presented by the

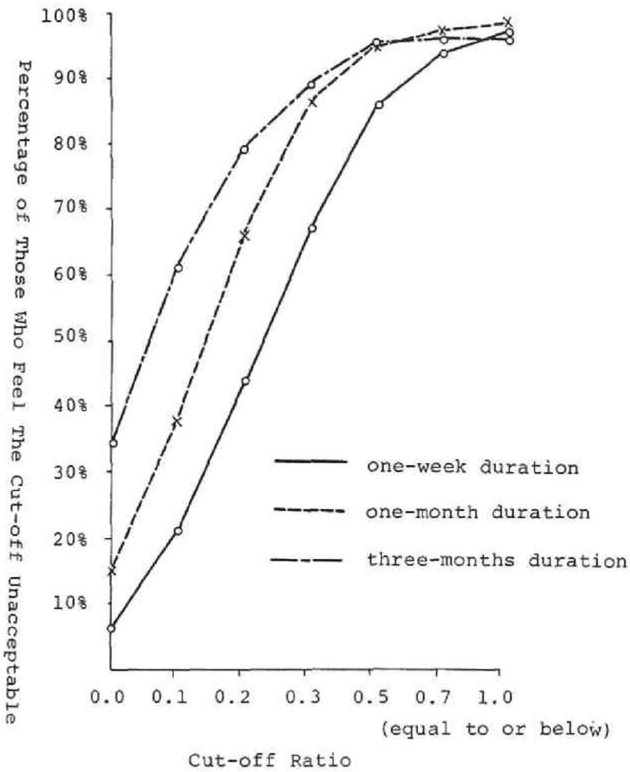


Fig. 7.4.1 Accumulated Numbers of Those who Feel Unacceptable to a Given Operation

Multidimensional Quantification Method developed by C.Hayashi. That is to say that the primary concern of the analysis is to find out those dominant factors which determine the drought-time behavioral pattern of the water users.

1) Multidimensional Quantification Method (II)

(a) Preliminary Discussion

Consider a questionnaire which contains R items corresponding to R kinds of questions. Each of the items, say item j, is assumed to consist of k kinds of choices (subcategories) from among which the answerer is required to select only one alternative. Here it is also postulated that all the items are non-scalable in the sense that any numerical value is not assigned by the answerer to each item, but only qualitative responses are identified from the dichotomous choices of the item. Furthermore we need to make another assumption that the questionnaire contains another question of different nature, which is set forth to conclude a set of the foregoing questions (items). This type of question is assumed to consist of l kinds of non-scalable choices out of which is selected a single choice by the answerer. We call this type of question the "outside criterion (variable)" or the "external criterion (variable)", and its relevant items the "strata". In this context the questionnaire is so designed that the stratum to which each of the answers belong can automatically be identified.

Now our concern is to obtain a functional relation in terms of statistico-mathematics between a given stratum to which a group of answers (samples) belong and their qualitative responses obtained from the dichotomous choices of the item. Once this is done, this statistico-mathematical model will provide us of an effective tool whereby the following examinations are available.

(i) to identify those important (dominant) items which are considered to have closer relation to a given stratum (pattern) than the rest of the items,

(ii) to infer the strata (pattern) of a given set of samples simply based on the information on the response characteristics reflected in each item.

In either sense of the word we can avail ourselves of this potential tool to "stratify" or "patternize" the samples' characteristics.

(b) Model Formulation

First the following notation is given.

$$\delta_i(jk) = \begin{cases} =1 \\ =0 \end{cases} \quad \cdots\cdots (7.4.1)$$

Then it follows:

$$\sum_{k=1}^{k_j} \delta_i(jk) = 1 \quad \cdots\cdots (7.4.2)$$

$$\sum_{i=1}^{n} \delta_i(jk) = n_{jk} \quad \cdots\cdots (7.4.3)$$

$$\sum_{k=1}^{k_j} \sum_{i=1}^{n} \delta_i(jk) = n \quad \cdots\cdots (7.4.4)$$

$$\delta_i(jk) \cdot \delta_i(jk') = \begin{cases} 0 & (k \neq k') \\ 1 & (k = k) \end{cases} \quad \cdots\cdots (7.4.5)$$

where n denotes the number of samples and n_{jk} the number of those samples who selected subcategory k in item j.

Here we introduce another kind of notation as follows.

$$\sum_{i=1}^{n} \delta_i(\ell m) \cdot \delta_i(jk) = f_{\ell m}(jk) \quad \cdots\cdots (7.4.6)$$

which leads to the following relations.

$$f_{\ell m}(jk) = f_{jk}(\ell m) \quad \cdots\cdots (7.4.7)$$

$$\sum_{m=1}^{m_\ell} f_{\ell m}(jk) = n_{jk} \quad \cdots\cdots (7.4.8)$$

Then we assume that the response pattern of sample i represented by a set of $\delta_i(jk)$ $(j=1,\cdots\cdots, R\,;\, k=1\,,\cdots\cdots, k_j)$ can be synthesized by a value of α_i which takes the form of linear combination of $\delta_i(jk)$. That is,

$$\alpha_i = \sum_{j=1}^{R} \sum_{k=1}^{k_j} \delta_i(jk) x_k \quad \cdots\cdots (7.4.9)$$

where $x_k (k=1, \cdots\cdots, k_j)$ denote the value of coefficients assigned to $\delta_i(jk)$.

Now our problem can be restated as the problem of determining the values of x_k in such a manner that the values of α_i well correspond to the stratum to which sample i belongs. In this connection, Hayashi introduced a hypothesis that the values of x_k be so determined that they may assure the largest (maximal) values of η which is the correlation ratio as given in the below.

$$\eta^2 = \frac{\sigma_b^2}{\sigma^2} \quad \cdots\cdots (7.4.10)$$

where σ_b^2 is the variance between strata, and σ^2 the total variance. They are given as follows.

$$\sigma^2 = \frac{1}{n} \sum_{i=1}^{n} \alpha_i^2 - \overline{\alpha}^2 \quad \cdots\cdots (7.4.11)$$

$$\sigma_b^2 = \sum_{t=1}^{T} \frac{n_t}{n} (\overline{\alpha}_t - \alpha)^2 \quad \cdots\cdots (7.4.12)$$

$$\overline{\alpha} = \frac{1}{n} \sum_{i=1}^{n} \alpha_i \quad \cdots\cdots (7.4.13)$$

$$\overline{\alpha_t} = \frac{1}{n_t} \sum_{i(t)=1}^{n_t} \alpha_i(t) \quad (7.4.14)$$

where n denotes the number of those samples, $i(t)$, belonging to stratum t.

The miximization of η^2 means the maximization of the effect of stratification, because the larger is η^2, the more homogeneous the response patterns of the samples belonging to each stratum, whereas the smaller becomes η^2, the more heterogeneous they turn to be. In this sense of the word, η^2 is considered a measure of discriminative power for stratification.

Accordingly, to maximize η^2 with respect to $x_{uv}(u=1, \cdots\cdots, R; v=1, 2, \cdots\cdots, k_u)$, we obtain

$$\frac{\partial \eta^2}{\partial x_{uv}}=0 \quad \cdots\cdots (7.4.15)$$

This is rewritten as

$$\frac{\partial \sigma_b{}^2}{\partial x_{uv}}=\eta^2 \frac{\partial \sigma^2}{\partial x_{uv}} \quad (u=1, \cdots\cdots, R; v=1, \cdots\cdots, k_u) \quad \cdots\cdots (7.4.16)$$

Since it can be shown that:

$$\frac{\partial \sigma_b{}^2}{\partial x_{uv}}=\frac{2}{n}\sum_j \sum_k \left[\sum_t \frac{g^t(jk) \cdot g^t(uv)}{n_t}-\frac{1}{n} n_{jk} n_{uv} x_{jk}\right] \quad \cdots\cdots (7.4.17)$$

and

$$\frac{\partial \sigma_b{}^2}{\partial x_{uv}}=\frac{2}{n}\sum_j \sum_k \left[f_{jk}(uv)-\frac{1}{n} n_{jk} n_{uv}\right] x_{jk} \quad \cdots\cdots (7.4.18)$$

where

$$g^t(jk)=\sum_{i(t)=1}^{n_t} \delta_{i(t)}(jk) \quad \cdots\cdots (7.4.19)$$

$$\sum_t g^t(jk)=n_{jk} \quad \cdots\cdots (7.4.20)$$

We can restate Equation (7.4.16) as follows.

$$\sum_{j=1}^{R} \sum_{k=1}^{k_j} h_{uv}(jk) x_{jk}=\eta^2 \sum_{l=1}^{R} \sum_{m=1}^{k_l}\left[f_{uv}(lm)-\frac{1}{n} n_{lm} n_{uv}\right] x_{lm} \quad \cdots\cdots (7.4.21)$$

where

$$h_{uv}(jk)=\sum_{t=1}^{T} \frac{g^t(jk) g^t(uv)}{n_t}-\frac{1}{n} n_{jk} n_{uv} \quad \cdots\cdots (7.4.22\text{-}1)$$

Alternately in terms of vector and matrix notations, the above relation of Equation (7.4.22-1) is rewritten as

$$HX=\eta^2 FX \quad \cdots\cdots (7.4.22\text{-}2)$$

where $H=\left[h_{uv}(jk)\right]$, $F=\left[f_{uv}(l_m)-\frac{1}{n} n_{lm} n_{uv}\right]$ and $X=\left[x_{11}, \cdots\cdots\cdots, x_{Rk_R}\right]'$

Accordingly our problem proved to be a class of eisen-value problem which can be solved by any one of the many elaborate techniques already developed.

It is commonly the case that the values of $x_{jk}(j=1, \cdots\cdots, R; k=1, \cdots\cdots, k_j)$ thus obtained are transformed into x'_{jk} for statistical convenience. That is,

$$x'_{jk}=x_{jk}-\sum_k x_{jk} n_{jk}/n \quad \cdots\cdots (7.4.23)$$

In this connection we shall set forth the notion of "range" which is defined for each item as the difference between the maximum and minimum values obtained from the set of $(x'_{j1}, \cdots\cdots\cdots, x'_{jk_j})$. Then the range of item i is meant to imply that it indicates the degree to which the item is concerned with the stratification (patternization) of the external variable.

external criterion	corresponding to the item of questionnaire	η	
recognized degree of water supply cut-off	You felt the water supply cut-offs were 1 very severe 2 rather severe 3 medium 4 rather moderate 5 very moderate	0.797	
candidate factors	**corresponding to the item of questionnaire**	**score**	**range**
quantity and pressure of water from the tap	You felt the water from the tap came out 1 sufficient 2 insufficient with low pressure 3 sometimes sufficient, otherwise no water 4 sometimes insufficient with low pressure, otherwise no water 5 no water all the time 6 otherwise	 -0.312 -0.376 -0.116 0.641 0.168 0.485	1.020
recognized percentage of water supply cut-off	You felt you were forced to reduce your usual water demand approximately by 1 100% 2 90% 3 80% 4 70% 5 60% 6 50% 7 40% 8 30% 9 20% 10 10% 11 0%	 0.109 0.118 0.148 -0.009 0.005 0.156 0.070 -0.031 -0.053 -0.013 -0.139	0.295

(1)

recognized duration time of cut-off operations	In your memory the cut-off operations lasted about 1 below 1 week 2 below 2 weeks 3 below 3 weeks 4 below 1 month 5 below 1.5 months 6 over 1.5 months	 -0.058 -0.077 -0.050 0.051 0.009 0.022	0.128
recognized changing pattern of cut-off operations	In your memory the cut-offs seem to be operated 1 initially severest, then gradually more moderate 2 severest in the middle of the period 3 initially most moderate, then gradually severer 4 always moderate 5 otherwise	 0.073 -0.038 0.099 -0.052 0.194	0.246
frustration coming from prolonged cut-off operations	You got blues 1 extremely 2 rather 3 a little 4 little 5 never as the cut-offs were prolonged	 0.008 0.122 -0.112 -0.133 -0.051	0.255
recognized level of save-water campaigns	You felt save-water campaigns worked 1 very effective 2 rather effective 3 not effective, nor ineffective 4 rather ineffective 5 very ineffective to lead yourself to voluntary curtailment	 0.440 0.446 -0.132 -0.023 -0.077	0.578

(2)

Table 7.4.3 Recognition Pattern Concerning Severeness of Cut-off Operations

external criterion	corresponding to the item of questionnaire	η	
willingness of the water users to save water	You reduced your water demands 1 rather willingly 2 unconsciously 3 rather unwillingly	0.795	
candidate factors	corresponding to the item of questionnaire	score	range
existence of wells	Do you have wells in your house ? 1 yes 2 no	 -0.008 0.006	0.014
quantity and pressure of water from the tap	You felt the water from the tap came out 1 sufficient 2 insufficient with low pressure 3 sometimes sufficient, otherwise no water 4 sometimes insufficient with low pressure, otherwise no water 5 no water all the time 6 otherwise	 0.083 -0.056 -0.045 0.070 -0.009 -0.052	0.139
recognized percentage of water supply cut-off	You felt you were forced to reduce your usual water demand approximately by 1 100% 2 90% 3 80% 4 70% 5 60% 6 50% 7 40% 8 30% 9 20% 10 10% 11 0%	 -0.002 0.106 0.055 0.090 0.096 -0.139 -0.091 -0.069 -0.087 -0.144 0.101	0.250

(1)

recognized duration time of cut-off operations	In your memory the cut-off operations lasted about 1 below 1 week 2 below 2 weeks 3 below 3 weeks 4 below 1 month 5 below 1.5 months 6 over 1.5 months	 -0.072 0.044 0.026 0.003 0.016 -0.020	0.116
recognized changing pattern of cut-off operations	In your memory the cut-offs seemed to be operated 1 initially severest, then gradually more moderate 2 severest in the middle of the period 3 initially most moderate, then gradually severer 4 always moderate 5 otherwise	 -0.031 0.039 0.036 -0.007 0.006	0.070
recognized level of the supplementary water supply service by wagons	In your memory you had supplementary water supply services by wagons 1 never 2 quite rarely 3 once every two days 4 once a day 5 more than twice a day	 -0.005 -0.020 0.058 0.065 -0.013	0.085
recognized level of save-water campaigns	You felt save-water campaigns worked 1 very effective 2 rather effective 3 not effective, nor ineffective 4 rather ineffective 5 very ineffective to lead yourself to voluntary curtailment	 0.040 0.031 -0.022 0.014 0.012	0.062

(2)

Table 7.4.4 Recognition Pattern Concerning Willingness to Save Water

2) Recognition Pattern Concerning Severeness of Cut-off Operations

In order to find out the dominant factors determining the recognition pattern concerning the severeness of cut-off operations, we set as the external variable the item of "recognized severeness of cut-off operations". Table 7.4.3 shows the calculation results for this case. As explained in the foregoing discussion, the theory underlying the quantification method tells us that the calculated range of each item (factor) stands for the degree of relevance to the patternization of recognized severeness. In light of this theory we examine the calculated range of each item as listed in Table 7.4.3. It shows that the item of the biggest range, i.e., the most influential factor, is "quantity and pressure of water from the tap", which is followed by "recognized cut-off ratio", and then by "recognized duration time of cut-off operations". In contrast to this those items (factors) proved to have smaller ranges such as "recognized changing pattern of cut-off operations", "recognized level of complementary water supply by wagons", and "recognized level of campaign activities". This means that those factors are less influential to the determination of "recognized severeness of cut-off operations".

Moreover comparison between those ranges and the scores assigned to those categories of each item, reveals whether or not the item has positive relevance to the patternization of "recognized severeness". To take an example of the item "quantity and pressure of water from the tap", we know that those who selected one of those categories, 1 "enough", 2 "a little with a low pressure" and 3 "sometimes enough, otherwise no water", are more likely to feel that the cut-off operations were not severe for them, than to feel otherwise. Conversely the rest of the water users who selected one of those categories, 4 "sometimes a little with a low pressure, otherwise no water" and 5 "no water all the time", tend to feel that the operations were rather severe for them. This kind of discussion will also apply to the other items.

3) Recognition Pattern Concerning Willingness to Save Water

In likewise as the above, we shall patternize the recognition pattern concerning willingness to save water, thereby analyzing what items (factors) are most influential to it. In applying the quantification method to the problem, the item "willingness of the water users to save water" is set as the external variable. The calculation results are listed in Table 7.4.4, the study of which shows the following:

(i) Those factors which are considered to be relatively influential to the patternization of "recognized willingness to save water", are 1 "recognized level of campaign activities", 2 "weariness about the prolonged duration of cut-off operations", and 3 "recognized cut-off ratio". On the other hand, the item "existence of wells" proved to be less influential to the "recognized willingness".

(ii) From this we may conclude that insofar as the water users who answered the questionnaire, or the samples of the survey in terms of statistics are concerned, there can be found such a conspicuous behavioral characteristic that the water users tend to take account of those influential items, consciously or unconsciously before they become willing or unwilling to spare water. It must be noted here that among those influential items, "recognized level of campaign activities" proved to be the most influential to the "willingness of the water users to spare water". This seems to vindicate our conclusion obtained in the preliminary discussion.

7.4.4 Summary and Discussion

The findings of our study are summarized:

(i) The recognized condition of cut-off operations corresponds to its actual condition insofar as its qualitative characteristics are concerned.

(ii) The extent to which the water users practised water-saving has much reliance on the recognized level of "save water" campaigns.

(iii) The water users become less willing to spare water as the duration time expands.

(iv) The recognition pattern concerning the severeness of cut-off operations is determined mainly by those factors such as 1 "quantity and pressure of water from the tap", and 2 "recognized cut-off ratio".

From this we may conclude that the mathematico-statistical analysis approach presented here provides a basic framework in which the studies to follow in 7.5 and 7.6, are performed. It also presents basic information on policy-making for acceptable operational controls in times of droughts.

In spite of such fruits obtained from our study, more study is needed to overcome the difficulties mainly involved in the technical problems of the design of questionnaire contents. That is to say that more sophisticated manner of questioning needs to be examined so as to make the answerer understand exactly what the answerer intends to mean.

7.5 Dynamic Programming Approach for Determining Optimal Policy for Drought-time Operational Controls

In this section our focus is shifted onto the problem of determining operational controls by some water supply agency. Basic to the problem is our assumption that water collection can be controlled by the agency, whereas it cannot control water demand, and that the term "operational control" exclusively means the control of water collection and that of water distribution is not explicitly treated here. In coping with this problem which will be further specified in the below, an attempt is made to build up a mathematical model which will be applied to the case study on Kyoto.

7.5.1 Description of the Problem

(i) Suppose a water supply-demand system which is conceptualized as consisting of single "reservoir" and single "water-user block". The term "reservoir" in this context means the totality of the water supply and distribution facilities which are operated by a water supply agency or "water supply sector" as will be referred to hereafter. Similarly "water user block" means the totality of those water users who owe their water supply to the water sector.

(ii) The term "water droughts" is defined here as a state where the storage of "reservoir" is depleted after a long continuation of secant rainfall, consequently leading to the inability to maintain the normal service level of water supply without cutting off "normal water supply". The definition of "normal water supply" is such that the amount of water supply which would be realized unless water droughts occurred. This term is used as synonymous to "normal water demand".

(iii) The droughts are predicted to continue for a certain period of time, which is referred to as "predicted duration period" or simply "duration period". The duration is divided into several stages.

(iv) Available amount of collected water which is conceptualized as the input to the reservoir changes with the passage of time. The changing pattern of maximal quantity is assumed to be predictable and given a priori.

(v) Available release from the reservoir which is conceptualized as the output of the reservoir assumed to be a control variable of the system.

(vi) Unless the potential water demand exceeds the storage in a given stage of the period, the draft is so operated that it equates the potential demand at that time. Otherwise the draft is so operated that it equates the storage at that time. That is to say that the reservoir is totally emptied if the potential water demand exceeds the storage.

7.5.2 Notation

x_c : reservoir volume

x_{i-1}: volume of remaining water in storage immediately after release in stage $i-1$ (or immediately before the initiation of stage i)

y_i : volume of water in storage immediately after replenishment (by collection) in the initiation of stage i

z_i : (available) amount of collected water in the initiation of stage i

ξ_i : (amount of) water supply which is equal to the amount of released water in stage i

q_i : (amount of) water demand in stage i

(q_i) : probability distribution function for q_i

7.5.3 Model Formulation

The volume of water in storage immediately after replenishment in the initiation of stage i, y_i, is formulated:

$$y_i=x_{i-1}+z_i \quad (i=1,\ 2,\ \cdots\cdots,\ n) \qquad (7.5.1)$$

The amount of water supply, ξ_i, which is equal to the amount of released water in stage i is determined by the following rule:

$$\xi_i=q_i \quad \text{for} \quad 0\leqq q_i<y_i \qquad (7.5.2)$$

$$\xi_i=y_i \quad \text{for} \quad q_i\geqq y_i\geqq 0 \qquad (7.5.3)$$

The volume of water in storage immediately after release in stage i, x_i, can be written as follows:

$$x_i=y_i-q_i \quad \text{for} \quad 0<q_i<y_i \qquad (7.5.4)$$

$$x_i=0 \quad \text{for} \quad q_i\geqq y_i\geqq 0 \qquad (7.5.5)$$

Additionally, there are constraints on the amount of replenished water:

$$0\leq y_i\leq x_c \qquad (7.5.6)$$

Then we set up the objective function which stands for the expected value of the degree of unacceptableness recognized by the water users in relation to the operational policies to be carried out with a given probability:

$$\text{Minimize}\ z=\sum_{i=1}^{n}\int_{q_i=\underline{q_i}}^{q_i=\overline{q_i}} P_i(r_i)\cdot\phi(q_i)\cdot dq_i \qquad (7.5.7)$$

where

$\overline{q_i}$: upper bound on the water demand with respect to stage i

$\underline{q_i}$: lower bound on the same above

$$r_i = \frac{q_i - y_i}{q_i} \quad \text{for} \quad q_i > y_i \qquad (7.5.8)$$

$$r_i = 0 \quad \text{for} \quad q_i \leq y_i \qquad (7.5.9)$$

r_i : cut-off ratio

$$p_i(r_i) = 0 \quad \text{for} \quad r_i = 0 \qquad (7.5.10)$$

$$0 \leq p_i(r_i) \leq 1 \quad \text{for} \quad r_i > 0 \qquad (7.5.11)$$

$p_i(r_i)$: penalty function with respect to r_i representing the degree of unacceptableness recognized by the water users.

7.5.4 Solution Technique

The mathematical model as formulated in the above has a multistage structure. One of the effective solution techniques which have already been developed and applied to different models of this type of mathematical structure, is dynamic programming developed by Bellman[7)8)]. We shall present a solution technique by use of dynamic programming.

Since dynamic programming requires to define the recurrence functions, let us rewrite the objective function to introduce them as follows.

$$z = f_n(x_0) = \underset{x_0 \leq y_0 \leq x_c}{\mathrm{Min}} \{ \int_{q_1=\underline{q_1}}^{q_1=\overline{q_1}} P_1(r_1)\,\phi(q_1)\,dq_1 + \int_{q_2=y_2}^{q_2=\overline{q_2}} f_{n-1}(0)\phi(q_2)\,dq_2 + \int_{q_2=\underline{q_2}}^{q_2=y_2} f_{n-1}(y_2-q_2)\phi(q_2)dq_2 \} \qquad (7.5.12)$$

Here the first term on the right-hand side of the above equation refers to the evaluation of the penalty function with respect to the first stage, the second and the third terms the integrated evaluation of those penalty functions with respect to the subsequent stages. The second term is equivalent to the statement that if in the first stage the water demand exceeded the upper limit to the suppliable amount of water, then it follows that the subsequent stages initiate with the condition that the volume of water in storage x_1, is equal to zero, with probability of $\phi(q_2)$. It also states that if the water demand happens to be lower than the upper limit to the suppliable amount of water, then it follows that the initial condition of the subsequent stages is so determined that the volume of water in storage x_1 equates the volume in storage immediately after replenishment y_2 less the amount of released water q_2 with probability of $\phi(q_2)$.

Similar discussion applies to the formulation of the general recurrence relation between a given stage and its subsequent stage.

$$f_i(x_{n-i}) = \underset{x_{n-i} \leq y_{n-i+1} \leq x_c}{\mathrm{Min}} \{ \int_{q_{n-i+1}=\underline{q_{n-i+1}}}^{q_{n-i+1}=\overline{q_{n-i+1}}} P_{n-i+1}(r_{n-i+1})\phi(q_{n-i+1})\,dq_{n-i+1}$$

$$+ \int_{q_{n-i+2}=y_{n-i+2}}^{q_{n-i+2}=\overline{q_{n-i+2}}} f_{i-1}(0)\phi(q_{n-i+2})\,dq_{n-i+2}$$

$$+\int_{q_{n-i+2}=\underline{q_{n-i+2}}}^{q_{n-i+2}=y_{n-i+2}} f_{i-1}(y_{n-i+2}-q_{n-i+2})\phi(q_{n-i+2})\,dq_{n-i+2}\} \qquad (7.5.13)$$

$$(i=2,\ \cdots\cdots,\ n)$$

where

$$f_1(x_{n-1}) = \underset{x_{n-1}\leq y_n\leq x_c}{\mathrm{Min}}\ \{\int_{q_n=\underline{q_n}}^{q_n=\overline{q_n}} p_n(r_n)\phi(q_n)\,dq_n\}. \qquad (7.5.14)$$

The recurrence relation thus obtained yields a theoretical basis for obtaining the sequence $f_i(x)$ inductively once $f_1(x)$ is known. We see that $f_1(x)$ determines $f_2(x)$, that $f_2(x)$ leads to an evaluation of $f_3(x)$, and so on. This procedure called the technique of dynamic programming assures a quick and accurate solution to the multistage (stochastic) model.

Additionlly for convenience sake of computation on a digital computer we shall replace the above recurrence relation of Equations (7.5.13) and (7.5.14) by the equations as follows.

$$f_i(x_{n-i}) = \underset{x_{n-i}\leq y_{n-i+1}\leq x_c}{\mathrm{Min}}\ \{\sum_{q_{n-i+1}=\underline{q_{n-i+1}}}^{q_{n-i+1}=\overline{q_{n-i+1}}} P_{n-i+1}(r_{n-i+1})\phi(q_{n-i+1})\Delta q_{n-i+1}$$

$$+\sum_{q_{n-i+2}=y_{n-i+2}}^{q_{n-1+2}=\overline{q_{n-i+2}}} f_{i-1}(0)\phi(q_{n-i+2})\Delta q_{n-i+2}$$

$$+\sum_{q_{n-i+2}=\underline{q_{n-i+2}}}^{q_{n-i+2}=y_{n-i+2}} f_{i-1}(y_{n-i+2}-q_{n-i+2})\phi(q_{n-i+2})\Delta q_{n-i+2}\} \qquad (7.5.15)$$

$$(i=2,\ \cdots\cdots,\ n)$$

$$f_1(x_{n-1}) = \underset{x_{n-1}\leq y_n\leq x_c}{\mathrm{Min}}\ \{\sum_{q_n=\underline{q_n}}^{q_n=\overline{q_n}} p_n(r_n)\phi(q_n)\Delta q_n\} \qquad (7.5.16)$$

7.5.5 Case Study on Kyoto City

1) Water Supply Problem of Kyoto City

Kyoto is characterized by a relatively affluent water source called "Sosui" or "Aqueduct from Lake Biwa". However population expansion and industrial development seem to be gradually menacing water scarcity crisis with Kyoto, especially in case the water level of Lake Biwa drastically drops from its normal level mainly due to a long duration of secant rainfall. As a matter of fact Kyoto experienced an unusual water drought in 1973 when it was forced to carry out "save water" campaigns. Additionally Kyoto is ready to provide necessary data for the model.

In light of these considerations we selected Kyoto as a study area.

2) Model Data

(a) Probability Distribution of Water Demand

By utilizing data on the actual amounts of water supply conducted from 1970 to 1974 except for 1973 when the drought in question occurred, the probability distributions of water demand were obtained as shown in Figure 7.5.1.

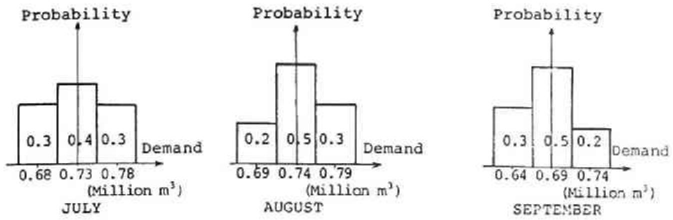


Fig. 7.5.1 Probability Distribution of Water Demand (Kyoto City)

(b) Penalty Function

We employ the function obtained in 7.4, representing the number of those who would not accept the operation insofar as the cut-off ratio is held constant at a given percentage or more than that for a limited duration time. (See Figure 7.4.1.)

(c) Calculation Cases

Before performing calculations, we preplanned the following analytical policies.

(i) Three different kinds of cases were established with respect to the duration time of the water-droughts. In one of them the duration time was set as the period ranging from (the beginning of) July to (the end of) August. In another case it was set as the period from August to September. In the last case it was set as the period from July to September.

(ii) Two different cases were established for two different "degrees of availability in collection" which are defined as the ratios of the total amounts of collected water over the period to the total amounts of potential water demand over the period. For a given degree of availability in collection, the assignment of available water to be collected is patternized according to the extent the water is assigned to each stage. This assignment pattern is termed "operation pattern" or simply "pattern" which represents how much water is collected for each stage.

In light of these policies we considered those six cases, thereby three different patterns being set a priori for Cases I-1 to I-4 and nine patterns for Cases II-1 to II-2, as listed in Table 7.5.1.

3) Calculation Results

Since similar results were obtained for Cases I-1 to I-4, we shall merely refer to the results of Cases II-1 and II-2.

(a) Case II-1

From the results as depicted in Figure 7.5.2 we understand:

(i) Irrespective of the initial storage, the most acceptable operational policy is such that relatively smaller amount of water is collected in the first stage, then increased amount of water in either of the subsequent two stages. (pattern 2)

(ii) In contrast to this the least acceptable policy irrespective of the initial level of storage is such that relatively larger amount of water is collected in the first stage, then it decreases with the passage of time, and the smallest amount of water in the last stage. (pattern 9)

(iii) Pattern 1 belonging to the type of operation similar to pattern 4 proved to be the second best alternative if and only if the initial storage exceeds three-fourths of the potential water demand, while it proved to be the least

(1)

		PATTERN	$Z_1:Z_2$		STAGE 1		STAGE 2	TOTAL	CASE NUMBER
DEGREE OF AVAILABILITY IN COLLECTION	0.2	1	1 : 2	JULY	Million m^3 0.393	AUGUST	Million m^3 0.787	Million m^3 1.18	I-1
		2	1 : 1		0.590		0.590	1.18	
		3	2 : 1		0.787		0.393	1.18	
	0.2	1	1 : 2	AUGUST	0.381	SEPTEMBER	0.763	1.14	I-2
		2	1 : 1		0.572		0.572	1.14	
		3	2 : 1		0.763		0.381	1.14	
	0.4	1	1 : 2	JULY	0.295	AUGUST	0.590	0.89	I-3
		2	1 : 1		0.443		0.443	0.89	
		3	2 : 1		0.590		0.295	0.89	
	0.4	1	1 : 2	AUGUST	0.286	SEPTEMBER	0.572	0.86	I-4
		2	1 : 1		0.429		0.429	0.86	
		3	2 : 1		0.572		0.286	0.86	

(2)

		PATTERN	$Z_1:Z_2:Z_3$	STAGE 1 JULY	STAGE 2 AUGUST	STAGE 3 SEPTEMBER	TOTAL	CASE NUMBER
DEGREE OF AVAILABILITY IN COLLECTION	0.2	1	1:2:4	Million m^3 0.247	Million m^3 0.494	Million m^3 0.987	Million m^3 1.73	II-1
		2	1:2:2	0.346	0.691	0.691	1.73	
		3	1:2:1	0.432	0.864	0.432	1.73	
		4	1:1:2	0.432	0.432	0.864	1.73	
		5	1:1:1	0.576	0.576	0.576	1.73	
		6	2:2:1	0.691	0.691	0.346	1.73	
		7	2:1:2	0.691	0.346	0.691	1.73	
		8	2:1:1	0.864	0.432	0.432	1.73	
		9	4:2:1	0.987	0.494	0.247	1.73	
	0.4	1	1:2:4	0.185	0.370	0.741	1.30	II-2
		2	1:2:2	0.258	0.519	0.519	1.30	
		3	1:2:1	0.324	0.648	0.324	1.30	
		4	1:1:2	0.324	0.324	0.648	1.30	
		5	1:1:1	0.432	0.432	0.432	1.30	
		6	2:2:1	0.519	0.519	0.258	1.30	
		7	2:1:2	0.519	0.258	0.519	1.30	
		8	2:1:1	0.648	0.324	0.324	1.30	
		9	4:2:1	0.741	0.370	0.185	1.30	

Table 7.5.1 Calculation Cases

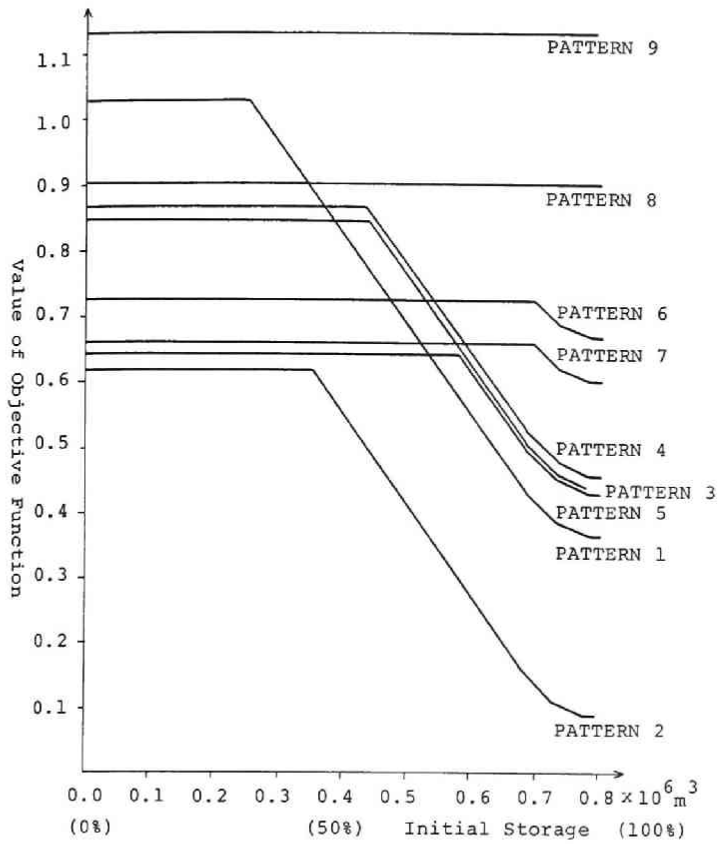


Fig. 7.5.2 Calculation Results of Case II-1

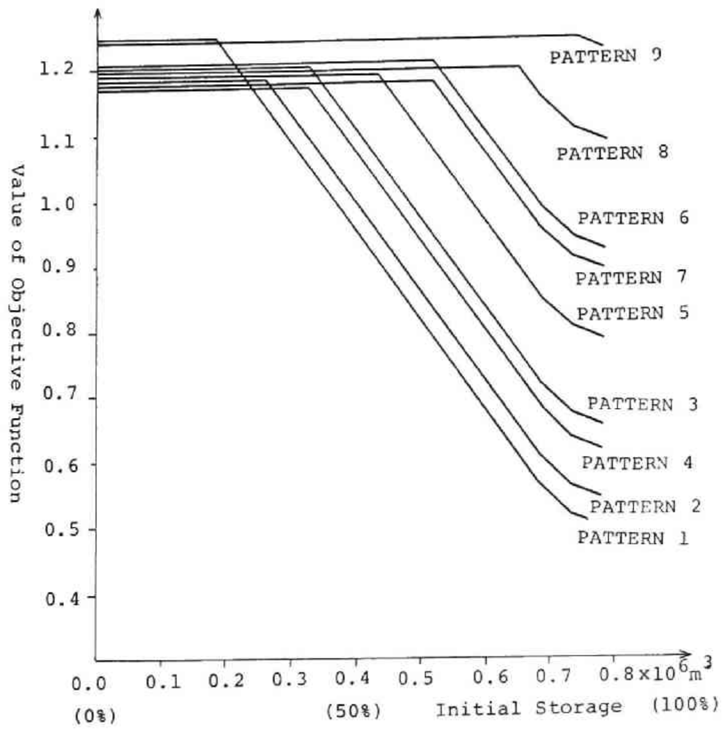


Fig. 7.5.3 Calculation Results of Case II-2

acceptable one next to pattern 9 if and only if the storage falls short of half of the demand.

(b) Case II-2

Figure 7.5.3 shows graphically the results of this case. From this we see:

(i) If the storage exceeds one-third of the potential water demand in the initial stage, the most acceptable policy is pattern 1. Pattern 2 which belongs to the same type of operations is the second best policy next to pattern 1. The difference between the two patterns of operations lies in that pattern 1 takes a more drastic changing pattern than pattern 2. Herewith attention needs to be paid to the fact that if the storage goes down from one-third of the demand, both of the two patterns become no more acceptable policy at all but fall in a category of the most unacceptable policies.

(ii) If the storage falls short of one-third of the demand, the most acceptable policy proved to be Pattern 4. This pattern belongs to the type of operation similar to patterns 1 and 2, the difference being that the former assumes less drastic changing pattern than the latter.

7.5.6 Summary and Discussion

The results of this part of the paper have indicated that:

(i) The problem of determining optimal policy for drought-time operational controls can be formulated as multi-stage stochastic programming to which dynamic programming is most efficiently applicable.

(ii) The "optimal" policy in this context means that it is an operation which is regarded as the most acceptable one to the water users.

(iii) The measurement of "acceptableness" in the above sense of the word can be performed by making use of the functional relationship obtained in 7.4.

But it must also be noted that there remain difficult problems left to be discussed from another viewpoint : ① whether or not it is reasonable to take such a qualitative factor as the objective function ; ② whether or not it is appropriate to measure such a qualitative factor by the manner presented here. In this connection more study is needed to overcome the difficulties.

The formulated system was based upon the assumption that although the total amounts of water is collected throughout the period limitedly from the water-source reservoir which was not explicitly included in the system, the choice of the optimal operational policy concerning the allocation of the total collected water in each stage of the period is under control of the water sector. This assumption inevitably precludes the study of such conditions that if the droughts are so severe that we cannot freely control collecting water any more.

Such being the case, our concern should be shifted from the control of collected water onto the controls of both water distribution and water demands. Let us now consider this type of problem in the next part of the paper.

7.6 System Dynamics Approach for the Analysis of Drought-time Water Supply-Demand Control Mechanism

This study treats the problem of drought-time water supply-demand control mechanism and present a systems dynamics approach to its analysis. To begin, we define the terminology which will be frequently found in this section and which provides basic idea underlying our study.[1)3)9)10)11)12)]

7.6.1 Terminology and Specification of the Problem

1) Water Droughts

To specify our definition of "water droughts", it will be called "extreme water droughts". The term "extreme" means that the droughts considered are so severe that controllability of collected water does not hold any more. But if otherwise specified, "extreme water droughts" are referred to simply as "water droughts".

2) Water Supply Control System (Mechanism)

We conceptualize the "water supply control system" as consisting of three components: ① collection, ② storage, and ③ distribution. It must be noted here that we assume that collection is uncontrollable. As a result impoundage of water by dams or reservoirs is given a priori. Then it follows that storage and distribution can be conceived as the major functions of the system. Here the storage is conceptualized as the total amount of water existing in water purification ponds and water transmission conduits as well as distributing reservoirs.

The time lag in transmission or distribution is neglected simply because our time scale for computation runs is selected as "day" as will be explained later.

3) Water Demand Control System (Mechanism)

The water demand system is a conceptualized system which describes the mechanism of the drought-time behavioral structure of the water users. It is assumed that the system is composed of four processes as follows.

(a) Potential Water Demand Generation Process

The "potential water demand" is defined as a "normal" water demand which would totally come out unless "water droughts" occurred. Here the term "normal is used to refer to the state where no drought is being encountered. Accordingly the "potential water demand generation process" describes the generation process of the "potential water demand".

(b) Water Users' Behavioral Response Process

It is assumed that the water users who would require the total amounts of the potential water demand are forced to spare some of them so as to adjust themselves to the water droughts. This process represents the following two processes:

(i) A process through which the water users are assumed to come to curtail their potential water demand to a certain extent in response to the "save-water" campaigns. This process will be referred to as the "economizing process".

(ii) A process through which the water users who get tired of cut-off operations are assumed to push up again their water demand closer to the potential water demand. This process will be referred to as the "anti-economizing process".

At this time the reader is required to recall that the setting of these two processes describing the water users' behavioral responses to cut-off operations is based on the results obtained in 7.4.

(c) Actual Water Demand Generation Process

The "actual water demand" is defined as a demand which is synonymous to the amount of water actually used by the water users and therefore synonymous to that actually supplied by the water supply sector. In this context the "actual water demand generation process" describes the process through which the amount of actual water demand generates and the amount of actual water supply is determined to meet it.

In this connection we shall explain the definition of the "water supply-demand gap". This term is defined as the difference in amount between the potential water demand and the actual water demand.

It needs to be added that the term "water demand sector" is used to mean collectively the water users.

7.6.2 Basic Structure of the Model

For limited space, the basic structure consiting of major feedback loops is depicted in Figure 7.6.1.

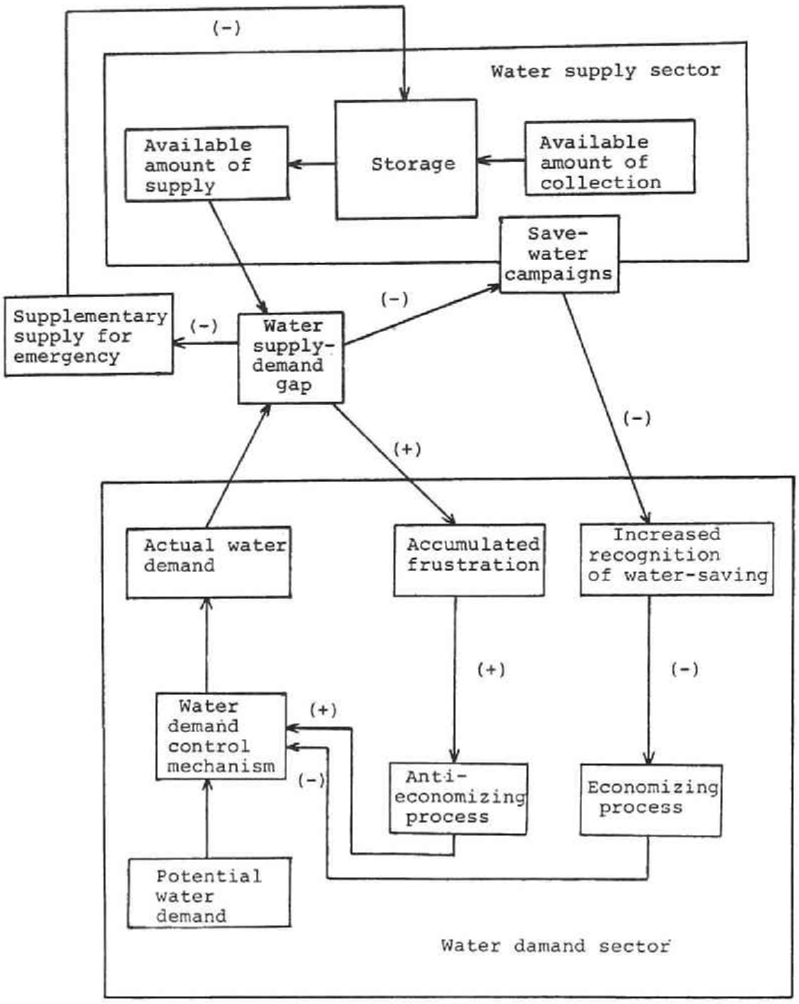


Fig. 7.6.1 Basic Structure of Model

7.6.3 System Dynamics Approach

In formulating the mechanism as outlined in this figure, a systems dynamics approach is attempted for the reasons as follows. (1) The basic structure of the mechanism to be incorporated in our model is characterized by several feedback loops which are mutually interrelated in a complicate manner. (2) Accordingly the system will behave against our intuitive expectations. (3) The performance of the mechanism varies with time. (4) In this context the system can be conceived as a dynamic system. (5) The incorporated structure that involves rather qualitative characteristics of the water users can hardly be formulated otherwise than system dynamics.

7.6.4 Study Area

Toyonaka City in Osaka was selected as the study area.

7.6.5 Model Formulation

1) Water Supply Control Mechanism

The Storage in the reservoir is expressed as:

SWR1.K = SWR1.J + (DT)(RG1.JK - RS1.JK) .... (7.L.1)

SWR2.K = SWR2.J + (DT)(RG2.JK - RS2.JK) .... (7.L.2)

RG.JK = RG1.JK + RG2.JK .... (7.A.1)

SWR1 : storage of water collected from the main water source (m^3)

SWR2 : storage of water collected from the minor water source (m^3)

RG1 : amount of collection from the main water source (m^3/day)

RG2 : amount of collection from the minor water source (m^3/day)

RG : total amounts of collection

Here the amount of collection which is assumed uncontrollable is given a priori by the table function as follows.

RG1.KL = TABLE(TABG, TIME.K,0,46,1) .... (7.R.3)

RG2.KL = (FIT)(YTAC.K) .... (7.R.4)

YTAC.K = TABLE(TAB5, TIME.K,0,46,1) .... (7.A.4)

YTAC : amount of supply by Osaka Prefectural Water Supply Agency (O.P.W.S.A.)

FIT : ratio of amount of collection from supply by O.P.W.S.A to YTAC

Here the number 46 represents 46 days which corresponds to the computation period.

The water demand is cut-off if the potential water supply-demand gap exists. This is written as follows.

SWL.K = MIN(WWD.K, SWR.K) .... (7.A.5)

SWR.K = SWR1.K + SWR2.K .... (7.A.6)

RS1.KL = (1 - COF.K)(SWL.K) .... (7.R.8)

RS2.KL = (COF.L)(SWL.K) .... (7.R.9)

COF.K = SWR2.K / SWR.K .... (7.A.7)

SWL : amount of actual water supply (m^3/day)

SWR : total amounts of storage of water (m^3)

WWD : reduced water demand (m^3/day)

COF : ratio of SWR2 to SWR

2) Water Demand Control Mechanism

The cut-off ratio is determined as follows:

RQCUT.K = (1/GWD.K)(GWD.K - SWL.K) .... (7.A.11)

RRQT.K = MAX(RQCUT.K,0) .... (7.A.12)

RQCUT : ratio of water deficiency

GWD : potential water demand. (m^3/day)

RRQT : cut-off ratio

Here the potential water demand is given a priori and set fixed during the entire period.

Campaign magnitudes are so patternized as:

PR.KL = TABLE(TABL, TIME.K, 0, 46, 1) .... (7.R.15)

(a) Ecconomizing Process

The repeated campaigns lead to an increase in the recognized effect of campaigns with a certain time lag. This process is expressed as:

LAB.K = LAB.J + (DT)(PR.JK - OUT.JK) .... (7.L.16)

OUT2.KL = LAB.K/DAM .... (7.R.17)

PATB.K = PAB.J + (DT)(OUT2.JK - OUT3.JK) .... (7.L.18)

OUT3.KL = PATB.K/DAX .... (7.R.19)

LAB : accumulation of campaign magnitudes

PATB : accumulation of recognized effects of campaigns

OUT2 : decreasing rate of LAB

OUT3 : decreasing rate of PATB

DAM : lag constant for LAB

DAX : lag constant for PATB

Here it is assumed that in both LAB and PATB are stored their inputs, i.e., PR and OUT2, respectively with a first-order expotential time lag as illustrated in Figure 7.6.2.

Then the accumulation of recognized effects of campaigns gives rise to actual curtailment in water-demand. This process is expressed:

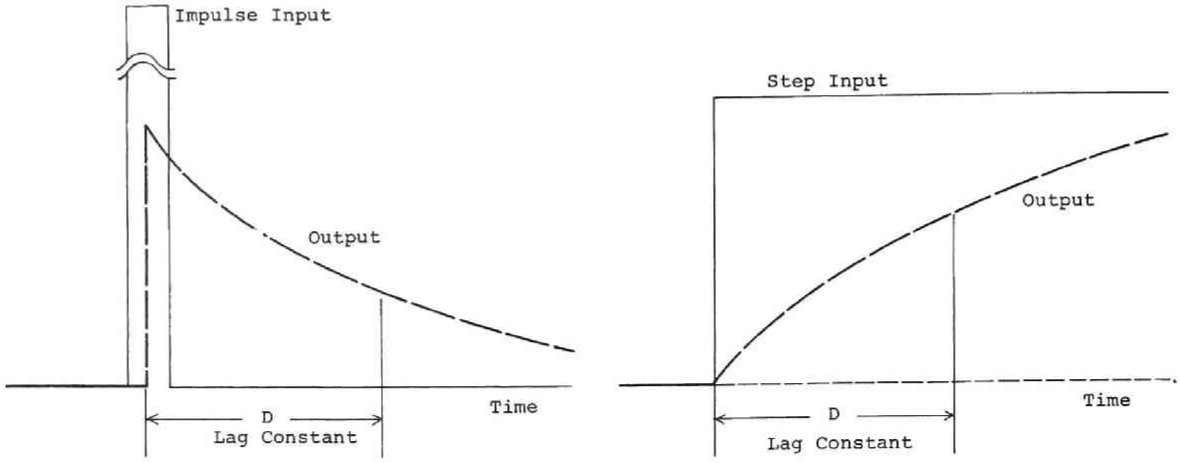


Fig. 7.6.2 1st-order Exponential Time Lag

SAVE.K = CLIP(AIR.K, SAVE1.K, TIME, K, 7) .... (7.A.20)
AIR.K = CLIP(SAVE3.K, SAVE2.K, TIME.K, 30) .... (7.A.21)
SAVE1.K = TABHL(TAB21, PATB.K, 0,1,0.1) .... (7.A.22)
SAVE2.K = TABHL(TAB22, PATB.K, 0,1,0.1) .... (7.A.23)
SAVW3.K = TABHL(TAB23, PATB.K, 0,1,0.1) .... (7.A.24)

SAVE : reduction ratio
AIR : dummy
SAVE1 : reduction ratio with respect to first one week
SAVE2 : reduction ratio with respect to first one month
SAVW3 : reduction ratio with respect to first three months

Here the table function (SAVE) of the reduction ratio for the accumulated value of the recognized effects of campaigns (PATB) is plotted in Figure 7.6.3 which was obtained from Figure 7.4.1.

(b) Anti-economizing Process

In formulating the mechanism of this process let us assume on the basis of the results obtained in 7.4, that the process consists of two parallel processes, i.e., 1 directly-anti-economizing process, and 2 indirectly-anti-economizing process. The former describes a mechanism such that the accumulated frustration of the water demand sector depending on the strength of cut-off operations (i. e., cut-off ratio) leads to an immediate push-up of its water demand. The latter is concerned with a mechanism such that the accumulated frustration of the water demand sector derived from the prolongation of cut-off operations will give another push-ups to its water demand after a certain time lag. Therefore two different push-ups are combined to increase the demand.

In this context the directly-anti-economizing process is expressed as follows.

ACUT.K = (WEEKLY.K)(RRQT.K) .... (7.A.25)
PUSH1.K = (CONST1)(ACUT.K) .... (7.A.26)
WEEKLY.K = TABHL(TAB8, TIME.K, 0, 90, 10) .... (7.A.27)
RDB.KL = CLIP(AUR.K, RDB1.K, TIME.K, 7) .... (7.R.20)
AUR.K = CLIP(RDB3.K, RDB2.K, TIME.K, 30) .... (7.A.28)
RDB1. K = TABHL(TAB11, RRQT.K, 0, 0.6, 0.1) .... (7.A.29)
RDB2. K = TABHL(TAB12, RRQT.K, 0, 0.6, 0.1) .... (7.A.30)
RDB3. K = TABHL(TAB13, RRQT.K, 0, 0.6, 0.1) .... (7.A.31)

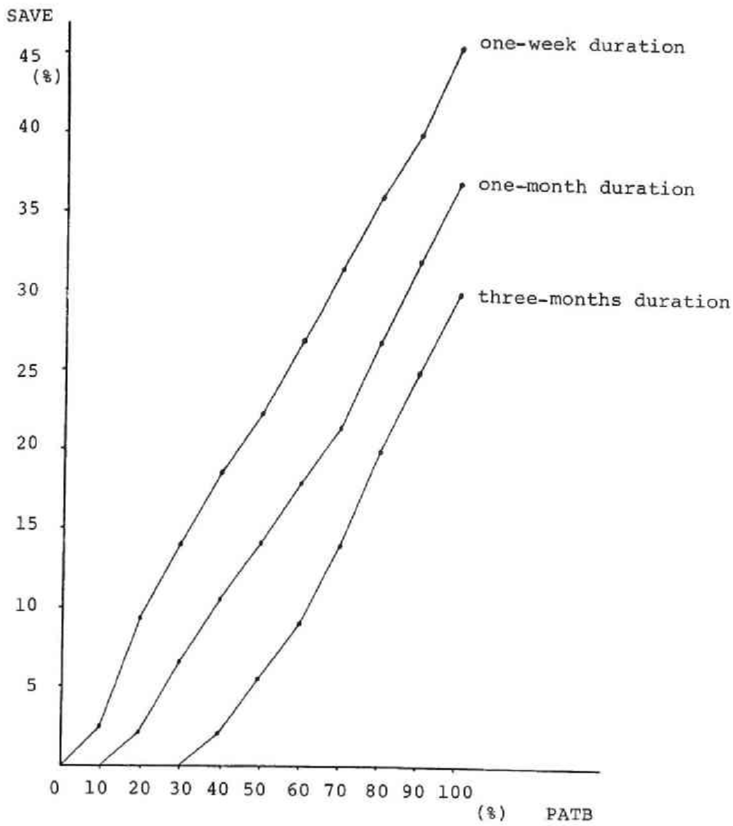


Fig. 7.6.3 Established Relation between PATB and SAVE

ACUT : accumulated frustration of the water demand sector depending on the strength of cut-off

PUSH1 : direct push-up ratio (ratio of directly-pushed-up amount to the potential water demand)

CONST1 : factor of transformation

WEEKLY : coefficient with respect to RRQT

RDB : dummy for a boxcar-train

RDB1 : dummies for table functions

RDB2 : same above

RDB3 : same above

AUR : dummy for a clip functions

The indirectly-anti-economizing process is expressed as follows.

DB = BOXLIN (7, 1) .... (7.B.32)

DB*1.k = DB*1.J + (DT)(RDB.JK - DB*1.J) .... (7.L.33)

SUDB.K = SUM1(7, DB.K) .... (7.A.34)

SSDB.K = (SUDB.K)(INT*1.K) / WEEK .... (7.A.35)

DB : accumulated frustration of the water demand sector due to prolongation of cut-off operations

DB*1 : dummy for a boxcar-train

SUDB : dummy representing the total amounts of the levels stored in the boxcar-trains

SSDB : average of DB over the past one week

WEEK : 7 days

INT*1 : dummy which takes on 1 for every seventh day, or 0 otherwise

The average of cut-off ratios over every one week is calculated as:

QAU = BOXLIN(7, 1) .... (7.B.36)

QAU*1.K = RRQT.K .... (7.A.37)

SUQU.K = SUM1(7, QAU.K) .... (7.A.38)

MEAN.K = (SUQU.K)(INT*1.K) / WEEK .... (7.A.39)

QAU : dummy for a boxcar-train

QAU*1: same above

SUQU : dummy representing the total amounts of the levels stored in the boxcar-trains

MEAN : average of cut-off ratio over every one week

The average of the accumulated frustration (SSDB) which is weigted by the average of cut-off ratio over every one week is transformed into the weighted average of accumulated frustrations. Then the weighted average which is accumulated with a certain time lag gives rise to the integrated push-ups of the water demand. This process is expressed:

VADB.KL = (MEAN.K)(SSDB.K) .... (7.R.40)

LAT.K = LAT.J + (DT)(VADB.JK - OUT.JK) .... (7.L.41)

OUT.KL = LAT.K / DAB .... (7.R.42)

PUSH2.K = (CONST2)(LAT.K) .... (7.A.40)

VADB : weighted average of SSDB
LAT : accumulated level of VADB
OUT : its decreasing rate
DAB : lag constant
CONST2 : factor of transformation

(c) Actual Water Demand Generation Process

Finally the reduction ratio is defined as follows.

RD.K = PUSH1.K + PUSH2.k .... (7.A.43)

Accordingly the reduced water demand is given by the following equations.

WD.KL = (GWD.K)(1 +RD.K) .... (7.R.45)

WWD.K = WWD.J + (DT)(WD.JK - WWD.J) .... (7.L.46)

WD : dummy
WWD : reduced water demand (m^3/day)

Now we obtained the outlined formulation of the model.

7.6.6 Simulation Results

1) Preliminary Discussion

Since our approach has stressed taking explicit account of the dynamic interaction between the water supply sector and water demand sector, our primary concern is with the questions that follow.

(i) Whether or not the assumed structure of the simulation model reflects well the existing mechanism governing over the drought-time water supply-demand controls.

(ii) Such being the case, which of the incorporated feedback loops plays an important role in balancing water supply and demand?

(iii) What are the inputs to the model which need to be given a priori and what about its outputs from which we may obtain some useful information on policy-making for the drought-time water supply-demand controls?

(iv) If there is a change in the assumptions, data inputs or policies, what impacts will it have on the outputs of the model?

The first question can be checked with a validity test by which the performance of the system (i.e., the outputs of the model) is set against the real-world data, and the discrepancy between the two is examined. If the discrepancy proves to be negligible in light of required accuracy for the model, we can conclude that the simulated model is a good representation of the real-world mechanism.

The questions of (ii) and (iv) can efficiently be approached by sensitivity experiments, which consist of making changes in the model, usually in the value of a particular parameter and compare the result simulated with the change to the result simulated without the change. This procedure is helpful in identifying those parameters or aspect of the model that could make significant differences in the outputs.

In light of these considerations the following simulation-run cases were prepared to make different kinds of experiments.

(i) The standard case is prepared by assigning to the inputs those actual values as were experienced by Toyonaka City in the selected drought-year of 1973. This is meant to say that the outputs are checked up with the actual values of the real-world data, in order to examine the validity of the model. (Case I, standard case)

(ii) Then to modify the structure of the model the following cases were prepared:

1 two different cases corresponding to two different time-lag constants for both the economizing and anti-economizing processes (Cases I and II-1);

2 two different cases corresponding to the two types of potential water demand patterns (Cases I and II-2);

3 one case where campaign magnitudes are extraneously patternized, and the other where its generation mechanism is incorporated into the model (Case II-3);

4 one case where a complementary water source for emergency is assumed to be non-existent, and the other where it is assumedly existent. (Case II-4).

(iii) To identify the impacts of a change in the imputs upon the outputs, the followings were prepared:

1 four different cases according to the difference in the campaign pattern (Cases I, III-1, III-2 and III-3);

2 three different cases according to the difference in the water-collection pattern (Cases I, III-4, III-5 and III-6).

Day	A*	B**	C***	B/A	C/A
0	178050	180000	180000	1.011	1.011
1	176560	180000	180000	1.019	1.019
2	179220	180000	180000	1.004	1.004
3	178510	179550	180000	1.006	1.008
4	175690	178740	180000	1.017	1.025
5	172160	178120	180000	1.034	1.046
6	176910	177650	180000	1.004	1.017
7	171070	177290	173960	1.036	1.016
8	164490	177100	174510	1.077	1.061
9	171510	171040	170140	0.997	0.997
10	171100	168940	168940	0.987	0.987
11	169840	165280	165280	0.973	0.973
12	165980	164960	164960	0.994	0.994
13	160300	164640	164640	1.027	1.027
14	159320	163840	163840	1.028	1.028
15	163910	164660	164660	1.005	1.005
16	155030	145440	145440	0.938	0.938
17	154950	145460	145460	0.939	0.939
18	159030	151310	151310	0.951	0.951
19	156370	153180	153180	0.980	0.980
20	151360	151080	151080	0.998	0.998
21	153650	137780	137780	0.897	0.897
22	152100	140290	140290	0.992	0.992
23	154380	139690	132500	0.905	0.858
24	155080	146120	131110	0.931	0.835
25	156950	142610	135270	0.909	0.862
26	152390	152080	139540	0.998	0.916
27	153720	147380	144570	0.959	0.940
28	152040	148660	150280	0.978	0.988
29	144550	145830	156670	1.009	1.084
30	131570	146150	163680	1.111	1.244
31	133340	147130	162110	1.103	1.216
32	138370	153230	153230	1.107	1.107
33	144850	155380	155380	1.073	1.073
34	143770	144700	144700	1.006	1.006
35	145010	150080	150080	1.035	1.035
36	151420	160190	160190	1.058	1.058
37	150880	161960	161960	1.069	1.069
38	152480	161870	161870	1.062	1.062
39	146470	160600	160600	1.096	1.096
40	139050	157230	157230	1.131	1.131
41	150390	151880	151880	1.010	1.010
42	142380	151610	151610	1.065	1.065
43	151990	151200	151200	1.062	1.062
44	155870	151420	151420	0.996	0.996
45	157830	147600	147600	0.935	0.935
46	147330	168680	168680	1.145	1.145

* actual values of SWL
** calculated values for SWL (standard case)
*** calculated values for SWL (another case)

Table 7.6.1 Validity Test

2) Model Validation

As explained above, the model's validity can be tested only by examining how well the model can produce past performance of the real world. As is obvious in Table 7.6.1, there seems to be a "negligible" discrepancy in the simulation outputs and real-world data in the sense that such extent of discrepancy can be regarded as inevitably attendant to our simulation runs if we allow for the accuracy in measuring our input data as well as the hypothesized structure of the model which were constructed on the basis of rather qualitative inspection of the real-world phenomena. From this we concluded that the simulation model can be considered a good representation of the real world, on the basis of which we can further conduct various experiments which seem to be otherwise impossible.

3) Standard Case

The outputs of this case are plotted in Figures 7.6.4, 7.6.5 and 7.6.6. At first blush the following can be easily understood.

(i) The amount of actual water supply (SWL) takes such a changing pattern quite similar to that of the total amount of collection (RG). This can be explained by the uncontrollability of collection which tends to lead to the uncontrollability of storage. Consequently the water supply demand gap can be adjusted only by reducing water demands indirectly (by doing "save-water" campaigns) or directly (by cutting off part of water supply).

(ii) A sharp rise or drop in the changing pattern of the cut-off ratio (RRQT) which is plotted in Figure 7.6.4 corresponds to the phase of an ill-balance in the amount of water between supply and actual demand.

(iii) This ratio (RRQT) changes with time in a manner quite contrary to that the amount of actual supply (SWL) changes. This can well be accounted for by the fact that the smaller SWL becomes, the more RRQT increases, and vice versa.

(iv) This seems to suggest that the macroscopic changing trend of RRQT is dependent on that of SWL. A closer examination of Figure 7.6.4 reveals, however, that the microscopic trend of RRQT is governed by that of the reduction ratio

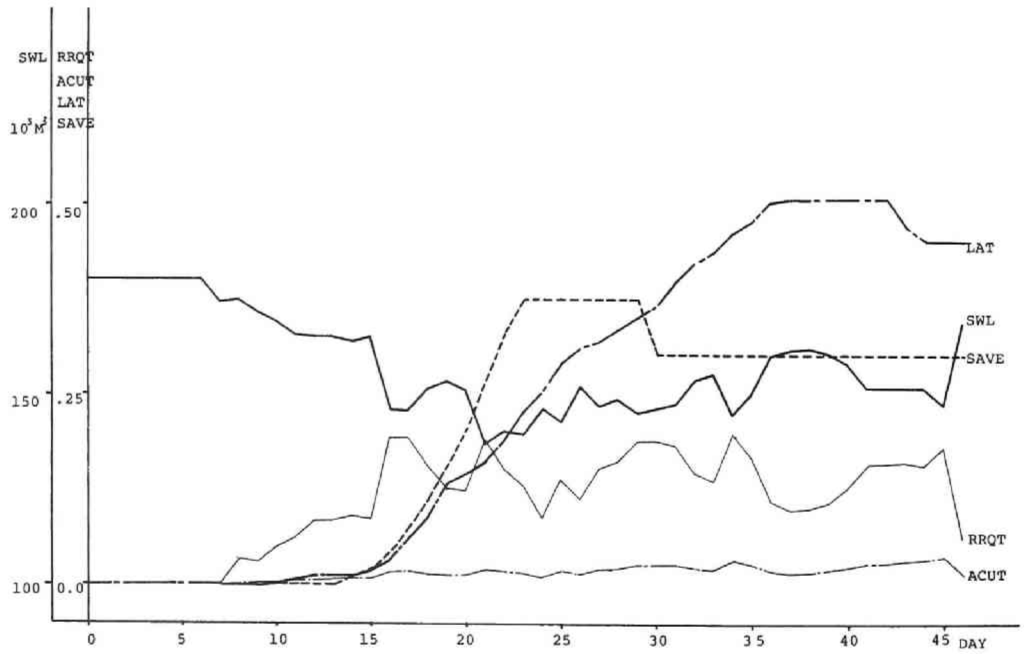


Fig. 7.6.4 Calculation Results (1) for Standard Case

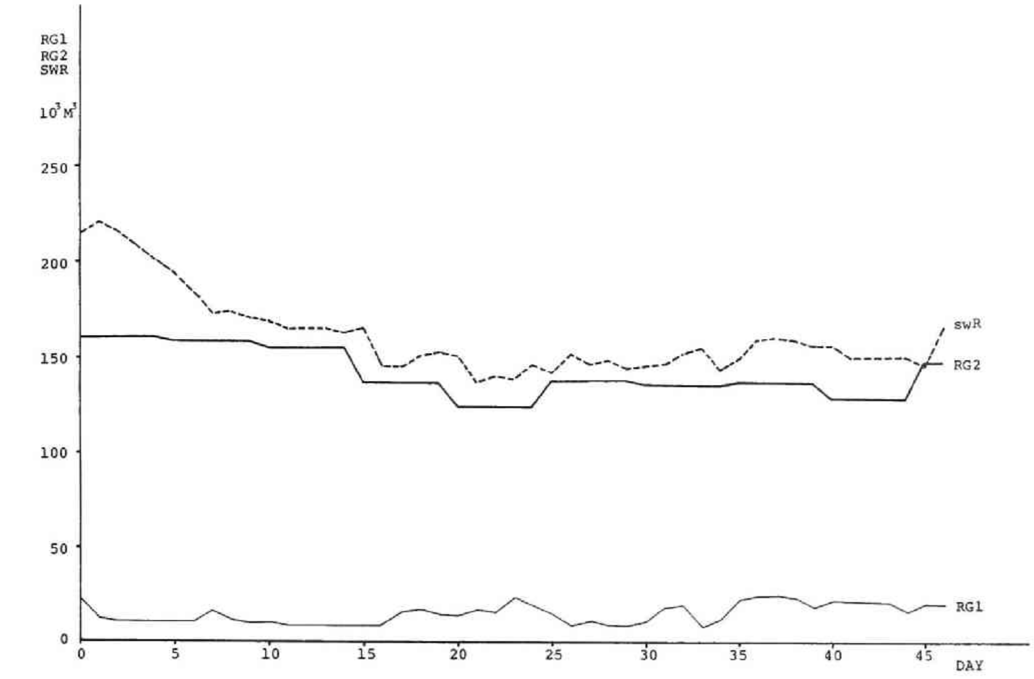


Fig. 7.6.5 Calculation Results (2) for Standard Case

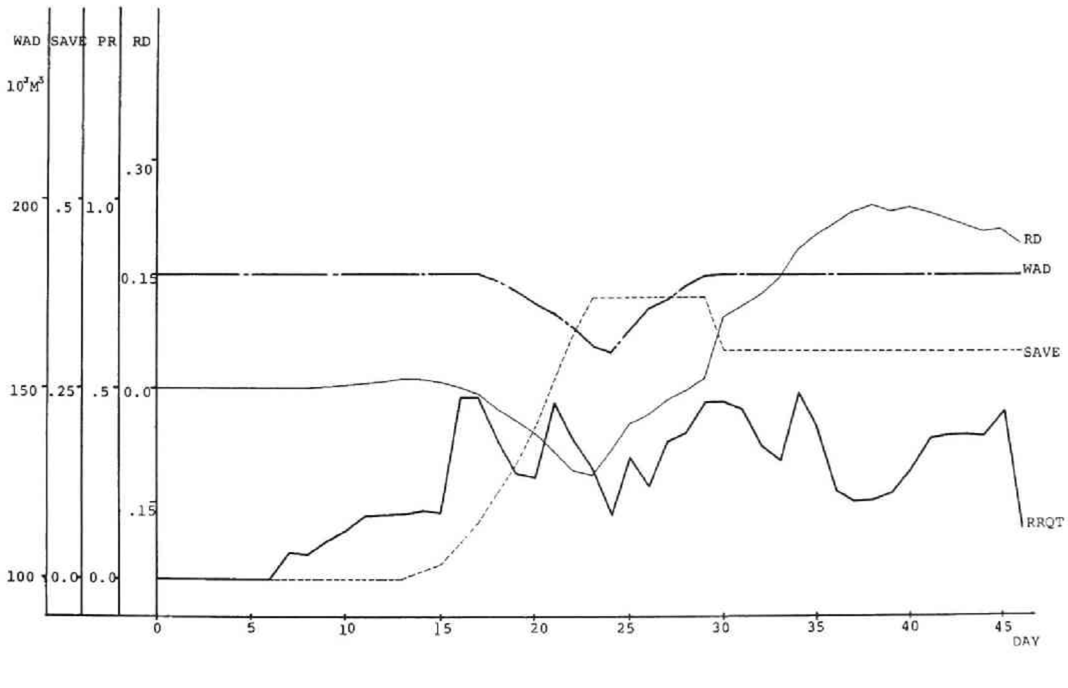


Fig. 7.6.6 Calculation Results (3) for Standard Case

(SAVE) and/or accumulated level of UADB(LAT). This is particularly the case for periods of 24th to 36th day and 40th to 45th in which RRQT sharply fluctuates and tends to decrease.

(v) Accordingly the combined changing pattern of the anti-economizing process which is obtained by superposing the above two patterns, LAT and SAVE, shows a pattern quite similar to that of RRQT. Especially from 15th to 23rd day, it takes a decreasing pattern, reflecting the effect of the foregoing save-water campaigns.

(vi) The model provides well-illustrated information with which deep insight into the mechanism treated can be obtained. (See Figures 7.6.4 and 7.6.6.)

4) Modified Cases

Mainly out of space consideration, we merely summarize the points.

(i) A slight change in the lag constant for LAB has little impact on the outputs. In case it largely changes, say from 7 days to 2 days, the results appear to differ much. But a closer inspection of the results reveals that so far as RRQT is concerned, there is slight difference between the two cases. This verifies that the assumed structure is qualitatively valid.

(ii) The calculation results for the case where Feedback Loops I and II (see Figure 7.6.1) are incorporated into the model, show that the patterns of the outputs for this case has a close resemblance to those for the standard case. In this context the mechanism incorporated seems to be valid in the sense that it is at least consistent with the standard case.

(iii) In the standard case the potential water demand was set fixed at 0.2 million m^3/day so as to take the average of the amounts of the water supplies conducted by the agency in the years of 1971 to 1974 (exclusive of 1973) when no cut-off operations were done. On the other hand in the case where it is assumed that the potential water demand obeys the normal distribution law with an average of 0.2 million m^3/day, Figure 7.6.7 shows that the changing patterns of the outputs differ to a considerable extent from those for the standard case. The most conspicuous difference is that the period of cut-off operations is cut down by some 35 days as compared to that for the standard case. This seems to be derived from it that the fall-down period of the potential water demand GWD happened to coincide with that of the available amount of supply SWL.

(iv) A different water-collection pattern leads to different patterns of the outputs. This can be explained by a heavy reliance of SWL on RG owing to the uncontrollability of the latter.

7.6.7 Summary and Discussion

The above findings seem to suggest that the assumed structure underlying the model, though invisible and intangible directly, proved to be valid enough in the sense that it well explains the real world and the results obtained from it are consistent with the actual phenomena. On this basis we can expect that the model provides an effective tool by which a deeper insight into the mechanism functioning in the drought-time water supply-demand controls can be obtained, and with which effective operational control policies can be examined when that situation occurs to which our assumptions adopted here roughly applies.

Lastly it should be added that the model is incomplete in that:

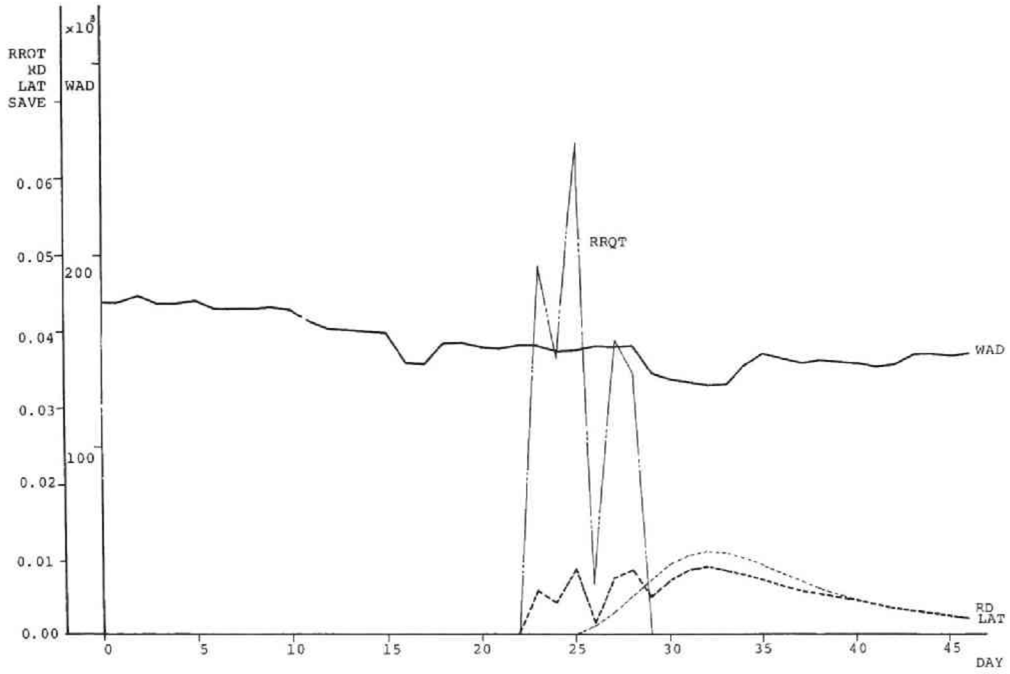


Fig. 7.6.7 Calculation Results for a Modified Case

(1) The parameters of the model applied to Toyonaka City, are identified on the basis of data collected in the survey of the drought-time behavioral characteristics of the water user of Tenri City in Nara Prefecture.[4)] But properly speaking the parametric identification should be performed on the basis of those data which exactly reflect the behavioral characteristics of the water users in Toyonaka, although it was attempted in vain, mainly because of limited data availability.

(2) The validity of the model needs to be more rigorously examined, because our approach attempted in this study is considered to guarantee merely that some of the necessary conditions for the validity of the model are satisfied in the constructed model. There seems to be two different approaches open to us. One approach is to go back to some basic statistical analyses of the drought-time behavioral characteristics of the water users. The other is to modify the structure of the model by utilizing those data which are warranted to reflect the behavioral characteristics of the water users in the study area.

7.7 Conclusion

In this chapter three approaches were presented to the analysis of drought-time water supply and demand control problems, which have scarcely been treated so far by researchers and practitioners concerned. Available approaches to this kind of unexplored fields of science seem to be classified into two. One is to initiate data-collecting works and to carry out very basic analyses of widespread dimensions of the problem. The other may be called a problem-finding approach whose primary objective is to demonstrate that a model, mathematical or simulation, constructed on the hypothesized structure, proves to be an effective tool to discuss how the mechanism should be controlled. As a matter of fact these two different approaches should be jointly performed, because the latter approach requires former approach to be carried out in advance, and the former is, more often than not, motivated by the latter. In this sense of the word either of the two approaches is complementary to each other, and should be combined in practice.

In view of the above considerations included in this paper are three studies. That is to say that the study presented in 7.4 whose approach falls into the first category, discussed a basic behavioral characteristics of the water users with specific reference to the periods of droughts, by making use of the Multi-dimentional Quantification Method. From this we could obtain basic information on some features of the drought-time behavioral characteristics of the water users. On this basis we explored two different studies, one included in 7.5, the other in 7.6. The approach adopted there can be classified into the second category.

The objective of the former study was to develop a mathematical model by which the optimal policy for the drought-time operational controls of water supply and demand has mathematically been analyzed.

In-depth examination of the assumptions adopted in either of the two studies reveals that though both concern the operational control problems, the study of 7.5 is mainly directed at the control of water collection by the water supply agency, while that of 7.6 deals with the interactive control mechanism functioning between the water supply and demand sectors, thereby controllability condition of water collection being replaced by that of both water distribution and demand. The latter study aimed first at analyzing the mechanism which are considered to control the water supply and demand in the times of droughts, and secondly at selecting effective operational policies concerned.

With much potentiality to be further developed, our systems approach for systematically analyzing the unexplored problem of the drought-time operational control has proven to be very effective and essential to the end of the analysis.

References

1) Okada, N., and Yoshikawa, K.: Systems Analysis of Operational Control of Water Supply and Use System in Drought-time, Proc. of JSCE (To Be Issued).

2) Wakatsuki, T.: Systems Analysis of Drought-time Operational Control Problems, Master Thesis Presented to the Faculty of Engineering, Kyoto University, March, 1975 (In Japanese).

3) Watanabe, H.: System Dynamics Simulation of Drought-time Water Supply and Use Controls, Graduation Thesis Presented to the Faculty of Engineering, Kyoto University, March, 1976 (In Japanese).

4) Ministry of Construction: Survey on Drought-time Water Use Characteristics; Case Study on Tenri City, Parts 1 and 2, 1974 (In Japanese).

5) Hayashi, C.: Multidimensional Quantification with the Application to Analysis of Social Phenomena, Annals of the Institute of Statistical Mathematics, Vol.5, No.2, 1954.

6) Hayashi, C.: On the Prediction of Phenomena from Qualitative Mathematico-statistical Point of View, Annals of the Institute of Statistical Mathematics, Vol.3, No.2, 1952.
7) Bellman, R.: Dynamic Programming, Princeton University Press, 1962.
8) Bellman, R.: Adaptive Control Processes, A Guided Tour, Princeton University Press, 1961.
9) Forrester, J.W.: Industrial Dynamics, The MIT Press, 1961.
10) Forrester, J.W.: Urban Dynamics, The MIT Press, 1969.
11) Yoshikawa, K.: Systems Analysis for Civil Engineers, Morikita Shuppan, Publ. Comp., 1975.
12) Yoshikawa, K., and Okada, N.: System Dynamics Approach to the Water Resources Management Related to the Regional Development, Proceedings Prepared for the 4th Pan Pacific Regional Conference, 1975.

Chapter 8 Conclusion

8.1 Summary

The objective of this paper has been to evolve a systems approach to the multi-faceted problem of area-wide, multi-modal water resources utilizations. The meaning of "water resources utilizations" has been narrowed to refer to the exploitation and use of water resources for water supply including its related regional activities. The notion of "area-wide" has been used to refer to some forms of extensive utilizations such as inter-basin streamflow deversions (inter-basin water transfers) and cross-boundary municipal and industrial water supplies. The term "multi-modal" ("dual-modal") means some forms of conjunctive utilization of both fresh water developments and wastewater reclamations.

8.1.1 Identification of the Problems Treated

By limiting the scope of this paper as such, the introductory chapter has explained the need for some systems approach to the above defined problem by ① gathering evidence on the importance of area-wide, multi-modal water utilizations; ② by illuminating the variety of system levels involved in the perceived problems of the area; and ③ by identifying the six kinds of studies. Thereby the high potential of the applicability of models, whether mathematical or simulation, was also pointed out by the author, and with this understanding the secondary objective of this paper was identified with the model-buildings and the developments of some effective solution algorithms for them.

8.1.2 Systems Analysis of the Water Resources Management on Regional Basis

Chapter 2 has treated the analysis of the impact of water resources management policies on the regional economy as well as those of the latter on the former with a view to provide the planner of this field with some basic information concerning: ① Is it not propitious to control the growth of population and/or industrial activities in case the available water resources are limited?; ② If the answer is yes, then in what occasions and to what extent?; ③ What the impacts on the region involved would be of some regulatory policies for the development of water resources? With this in mind, a systems analysis has been performed to study the problem by virtue of systems dynamics and with a case study on the two adjoining regions, the Tohban and Hokusetsu Regions —— both located in the highly industrialized area (called the Hanshin Industrial District) of Hyogo Prefecture in Japan.

The analysis of the simulation runs has revealed:

(i) As far as the Tohban and Hokusetsu Regions are concerned, the water resource management has been seen to have much impact on the regional development, especially in the earlier stages of the period, say in 10 or 20 years, on the assumption that relatively efficient water use patterns coupled with the intensified developments of water resources could not be attained.

(ii) Otherwise if such kind of intensified developments can be practiced, the population or industrial land area eventually reaches its upper limit, thereby resulting in the slowdown of the growth of the water demand.

(iii) Is follows from these findings that the development of the regions concerned will be, early or late, confronted with either of the two problems:

shortages of water or land —— either of which would restrict further growth of the regions.

8.1.3 Systems Analysis of Intra-basin, Multi-modal Water Utilization System

Chapters 3 to 5 are based on the assumption that the future development pattern of the regional economy is given a priori. Then the author's attention has been turned to the three features of the problem related to the development and utilization of some specified water resource systems, i.e., intra-basin and inter-basin systems.

To begin with, Chapter 3 has treated the analysis of the intra-basin development systems.

The intra-basin system that utilizes water within the boundary of a given single basin is considered a geographical unit of water supply and use.

The major findings of the case study on the basin of the Kakogawa River which runs across the part of Hyogo Prefecture could be summarized as follows.

(i) The implementation of the reclamation system is found to be concentrated on the downstream zone where the entire demands for industrial uses are estimated exclusively higher than the other zone.

(ii) The needed amount of renovated water decreases roughly in proportion to the increase in the amount of available fresh water to be developed in the headwaters.

(iii) The associated costs decrease with the increase in the available amount of fresh water. But if the cost for developing fresh water is included into the associated costs, the total costs tend to increase as larger amounts of fresh waters are developed.

(iv) When the quality standards are elevated to higher ones, a more extensive scale of reclamation system is demanded.

(v) The method of feasible directions combined with the penalty method due to Fiacco and McCormick has proven to be effective to the solution search of the presented nonlinear model.

(vi) This kind of intra-basin development problem should receive increased attention in prior to and together with the analysis of some inter-basin development.

8.1.4 Systems Analysis of Integrating Process of Inter-basin, Multi-modal Water Utilization System

On the basis of Chapter 3, our next concern has been shifted from the intra-basin system to some form of inter-basin system with a conjunctive use of the water reclamation system. Thereby on the basis of the observation that both systems are implemented and managed by different planning bodies, the author's incidental interest has been identified with the analysis of the integrating process of the two different planning functions involved. With this intention the problem has been further specified and modelled by use of linear programming and the decomposition principle. The close scrutiny of the results derived from the case study on the southern part of Hyogo Prefecture has led the author to the observation that:

(i) The reclamation system should be implemented exclusively in those regions where relatively high water demands are estimated.

(ii) The parametric programming analyses in which either of the associated costs or the estimated water demands are parameterized, have revealed that the overestimated implementation costs and/or the overestimated water demands tend to place the reclamation system in a more advantageous position.
(iii) The decomposition-based analysis has revealed that the earlier stages of the integrating process correspond to the phase of roughing out both the total amounts of the water supply to be covered by the fresh water development system and those shared by the reclamation system. This implies that if such a roughing-out mechanism is given primary consideration in prior to the detailed examination of the optimal global system it would reduce a vast amount of work needed to follow it.

8. 1. 5 Systems Analysis of the Coordinated Attainment of Multi-goal, Inter-basin, Multi-modal Water Utilization System

Chapter 5 has dealt with the inter-basin, multi-modal water resource development system from another point of view. Its primary objectives have been: 1 to identify the major goals of the system; 2 to illustrate the need of the coordination of the goals involved; 3 to convert the problem to a mathematical programming problem which can be modelled by virtue of the nonlinear goal programming method.

The major findings of the case study on the same region as in Chapter 4 are summarized.
(i) The modified nonlinear programming developed by the author has proven to yield a very reasonable solution after a certain number of iterations. It has also been found that the nonconvexities involved in the nonlinear goal constraints can be overcomed in this manner.
(ii) The augmentation of streamflows by promoting the inter-basin streamflow diversion system has been found effective for the purpose of quality alleviations to certain extent, but a higher quality has been proven effectively attainable chiefly by promoting the reclamation system.
(iii) It has also been shown that if one takes a demand-dependent alternative, a balanced mix of the two alternative systems are required to be implemented, whereas if one seeks for a demand-regulatory alternative, increased coverage ratio of the total demands by the fresh water system seems to be efficient.

8. 1. 6 Analysis and Design of Water Distribution System

Two topics covered in Chapters 6 and 7 have concerned the water supply and distribution system —— an extremity sub-system of the total water utilization system. With this view the central question of Chapter 6 have been the optimal implementation of the distribution system.

Close examination of the results of the application studies has revealed:
(i) The mathematical model presented in this study provides some basic frameworks for both the design and the planning of the water distribution system.
(ii) Some important guidelines in which to design some distribution system have been obtained.
(iii) The cross-boundary water distribution system has been found more advantageous than isolated systems, judged from the viewpoints of economy and hydraulic conditions.

8.1.7 Systems Analysis of Drought-time Operational Control of Water Supply and Use System

Chapter 7 has presented three approaches to the analysis of water supply and use system, which has scarcely been treated so far by researchers and practitioners. The first study has discussed a basic behavioral characteristics of the water user with specific reference to the periods of droughts, by use of the Multidimensional Quantification Method. From this we could obtain basic information on some features of the drought-time behavioral characteristics of the water users. The objective of the second study has been to develop a mathematical model by which the optimal policy for the drought-time operational controls of water supply and demand could be operationally considered. The last study has aimed at (1) analyzing the mechanism which functions to control the water supply and demand in times of droughts, and (2) selecting effective operational policies concerned. Analysis of the computation runs of the case study on Toyonaka City has shown that the model presents an effective tool by which a deeper insight can be given into the mechanism functioning in the drought-time water supply-demand controls, and with which effective operational control policies can be examined.

8.2 Overall Assessment of the Presented Approach

8.2.1 Potential of Systems Approach

If the problem is confined to the exploitation and use of water resources for water supply as has been done in this paper, the presented systems approach has been found to serve quite effectively the purpose of presenting some well-structured information to find our future courses of action for tackling the related complex problem. The author believes that (1) this paper has illustrated both the need and way to apply a systems approach to the problem and (2) it has thrown light on the potential of different levels of systems analyses as tools to present the evaluated material in the form of relevant information to the planner, designer, or manager of the related fields.

Though the author's conclusions point to a vital need for this kind of systems approach to the specified problem, it seems also clear that the studies included in this paper is not complete nor exhaustive. They are broken down into three categories; (1) much room to be modified and sophisticated; (2) further extensive scope of the problem uncovered; (3) limitations of systems approach. The first and third points have already been touched upon as research needs in each of the chapters. At the final stage of this paper the second point is briefly explained in the following paragraphs.

8.2.2 Problems Uncovered

Many important water problems have not been at stake in this paper. They include flood control, power generation, navigation, recreation, etc. This paper has given them only passing consideration before limiting the scope of study to the problem of water supply. Furthermore political, administrative and institutional questions involved in the water resources management have been untouched in this paper. All of them have been excluded not because they are less important, but because they are too complicate and multi-faceted to be covered by this paper. The author understands that same kind of systems approaches would well serve for the purpose of analyzing the problem.

Though there is much room to be developed and different approaches could be applied, the author believes that the presented model is considered very effective to cope with the specified problem.